CHAPTER 3
EVALUATION OF SETTLEMENT FOR STATIC LOADS

3-1. General. This chapter presents the evaluation of immediate settlement in cohesionless and cohesive soils and consolidation settlement of soil for static loads. Settlement is denoted as a positive value to be consistent with standard practice.

3-2. Components of Settlement. Total settlement ρ in feet, which is the response of stress applied to the soil, may be calculated as the sum of three components

$$\rho = \rho_i + \rho_c + \rho_s \tag{3-1}$$

where

ρ_i = immediate or distortion settlement, ft
ρ_c = primary consolidation settlement, ft
ρ_s = secondary compression settlement, ft

Primary consolidation and secondary compression settlements are usually small if the effective stress in the foundation soil applied by the structure is less than the maximum effective past pressure of the soil, paragraph 1-5a.

a. Immediate Settlement. Immediate settlement ρ_i is the change in shape or distortion of the soil caused by the applied stress.

(1) Calculation of immediate settlement in cohesionless soil is complicated by a nonlinear stiffness that depends on the state of stress. Empirical and semi-empirical methods for calculating immediate settlement in cohesionless soils are described in Section I.

(2) Immediate settlement in cohesive soil may be estimated using elastic theory, particularly for saturated clays, clay shales, and most rocks. Methods for calculating immediate settlement in cohesive soil are described in Section II.

b. Primary Consolidation Settlement. Primary consolidation settlement ρ_c occurs in cohesive or compressible soil during dissipation of excess pore fluid pressure, and it is controlled by the gradual expulsion of fluid from voids in the soil leading to the associated compression of the soil skeleton. Excess pore pressure is pressure that exceeds the hydrostatic fluid pressure. The hydrostatic fluid pressure is the product of the unit weight of water and the difference in elevation between the given point and elevation of free water (phreatic surface). The pore fluid is normally water with some dissolved salts. The opposite of consolidation settlement (soil heave) may occur if the excess pore water pressure is initially negative and approaches zero following absorption and adsorption of available fluid.

(1) Primary consolidation settlement is normally insignificant in cohesionless soil and occurs rapidly because these soils have relatively large permeabilities.

(2) Primary consolidation takes substantial time in cohesive soils because they have relatively low permeabilities. Time for consolidation

increases with thickness of the soil layer squared and is inversely related to the coefficient of permeability of the soil. Consolidation settlement determined from results of one-dimensional consolidation tests include some immediate settlement ρ_i . Methods for calculating primary consolidation settlement are described in Section III.

c. Secondary Compression Settlement. Secondary compression settlement is a form of soil creep which is largely controlled by the rate at which the skeleton of compressible soils, particularly clays, silts, and peats, can yield and compress. Secondary compression is often conveniently identified to follow primary consolidation when excess pore fluid pressure can no longer be measured; however, both processes may occur simultaneously. Methods for calculating secondary compression settlement are described in Section IV.

Section I. Immediate Settlement of Cohesionless Soil For Static Loads

3-3. Description of Methods. Settlement in cohesionless soil (see paragraph 1-5c for definition) is normally small and occurs quickly with little additional long-term compression. Six methods described below for estimating settlement in cohesionless soil are based on data from field tests (i.e., Standard Penetration Test (SPT), Cone Penetration Test (CPT), Dilatometer Test (DMT) and Pressuremeter Test (PMT). Undisturbed samples of cohesionless soil are normally not obtainable for laboratory tests. The first four empirical and semi-empirical methods - Alpan, Schultze and Sherif, Modified Terzaghi and Peck, and Schmertmann approximations - were shown to provide estimates from about 1/4 to 2 times the measured settlement for 90 percent confidence based on the results of a statistical analysis (item 27). Penetration tests may not be capable of sensing effects of prestress or overconsolidation and can underestimate the stiffness that may lead to overestimated settlements (item 37).

a. Alpan Approximation. This procedure estimates settlement from a correlation of (SPT) data with settlement of a 1-ft square loading plate. The settlement of a footing of width B in feet is (item 1)

$$\rho_i = m' \bullet \left[\frac{2B}{1 + B}\right]^2 \bullet \frac{\alpha_o}{12} \bullet q \qquad (3\text{-}2)$$

where

ρ_i = immediate settlement, ft
m' = shape factor, $(L/B)^{0.39}$
L = length of footing, ft
B = width of footing, ft
α_o = parameter from Figure 3-1a using an adjusted blowcount N' from Figure 3-1b, inches/tsf
q = average pressure applied by footing on soil, tsf

(1) Blowcount N . N is the average blowcount per foot in the stratum, number of blows of a 140-pound hammer falling 30 inches to drive a standard sampler (1.42" I. D., 2.00" O. D.) one foot. The sampler is driven 18 inches and blows counted the last 12 inches. The blowcount should be determined by ASTM Standard Test Method D 1586. Prior to 1980 the efficiency of the hammer was not well recognized as influencing the blowcount and was usually not considered in analysis.

(a) The measured blowcounts should be converted to 60 percent of input energy N_{60} by

$$N_{60} = N_m \bullet \frac{ER_i}{60} \tag{3-3a}$$

$$ER_i = \frac{E_i}{E^*} \tag{3-3b}$$

where

N_{60} = blowcounts corrected to 60 percent energy ratio
N_m = blowcounts measured with available energy E_i
ER_i = measured energy ratio for the drill rig and hammer system
E^* = theoretical SPT energy applied by a 140-pound hammer falling freely 30 inches, 4200 inch-pound

(b) The converted blowcount N_{60} is entered in Figure 3-1a with the calculated effective overburden pressure σ_o' at the base of the footing to estimate the relative density D_r . The relative density is adjusted to 100 percent using the Terzaghi-Peck curve and the adjusted blow count N' read for D_r = 100 percent. For example, if σ_o' = 0.3 tsf and N = 10 , then the relative density D_r = 67 percent, Figure 3-1a. The adjusted N' is determined as 31 for D_r = 100 percent.

(2) Parameter α_o . The adjusted blowcount is entered in Figure 3-1b to determine α_o . α_o = 0.1 inch/tsf for adjusted N' = 31.

b. Schultze and Sherif Approximation. This procedure estimates settlement from the blowcount of SPT results based on 48 field cases (item 60)

$$\rho_i = \frac{f \bullet q \bullet \sqrt{B}}{N_{ave}^{0.87} \bullet \left(1 + 0.4\dfrac{D}{B}\right)} \tag{3-4}$$

where

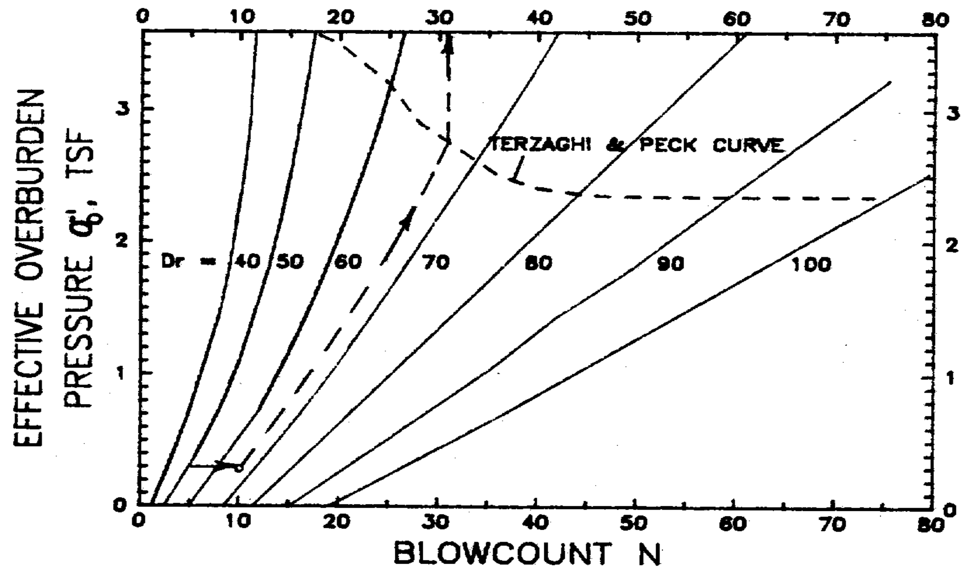
f = influence factor from elasticity methods for isotropic half space, Figure 3-2
H = depth of stratum below footing to a rigid base, ft
D = depth of embedment, ft
N_{ave} = average blowcount/ft in depth H

The depth to the rigid base H should be ≤ 2B. N_{ave} is based on measured blowcounts adjusted to N_{60} by Equations 3-3.

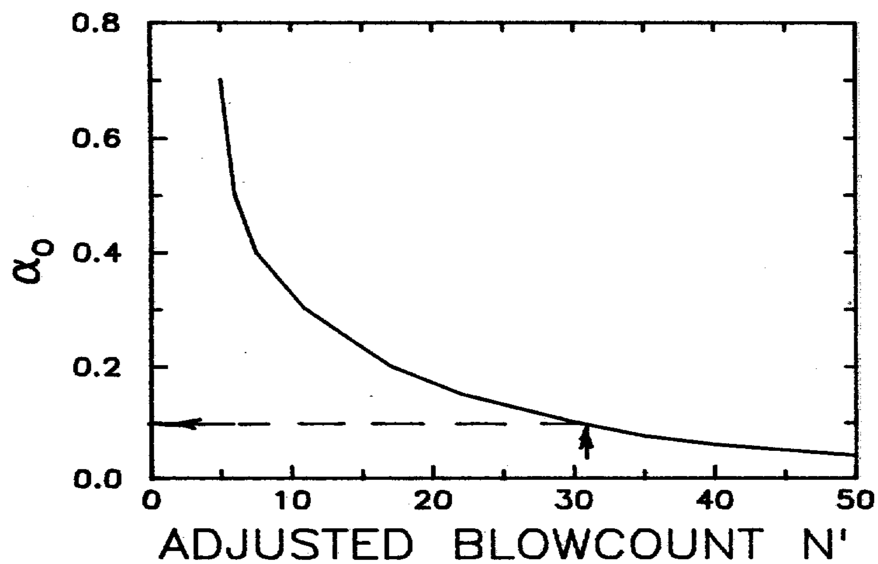
c. Modified Terzaghi and Peck Approximation. This procedure is a modification of the original Terzaghi and Peck approach to consider overburden pressure and water table (items 50,51)

$$\rho_i = \frac{q}{18 \bullet q_1} \tag{3-5}$$

where q_1 = soil pressure from Figure 3-3a using corrected blowcount N' and the ratio of embedment depth D to footing width B , tsf. The corrected blowcount N' is found from



a. ADJUSTED BLOWCOUNT FROM N AND σ_o'



b. PARAMETER α_o FROM ADJUSTED BLOWCOUNT

Figure 3-1. Chart to apply Alpan's procedure (data from item 1)

$$N' = N \bullet Cw \bullet Cn \tag{3-6}$$

where

N = average blowcount per foot in the sand
Cw = correction for water table depth
Cn = correction for overburden pressure, Figure 3-3b

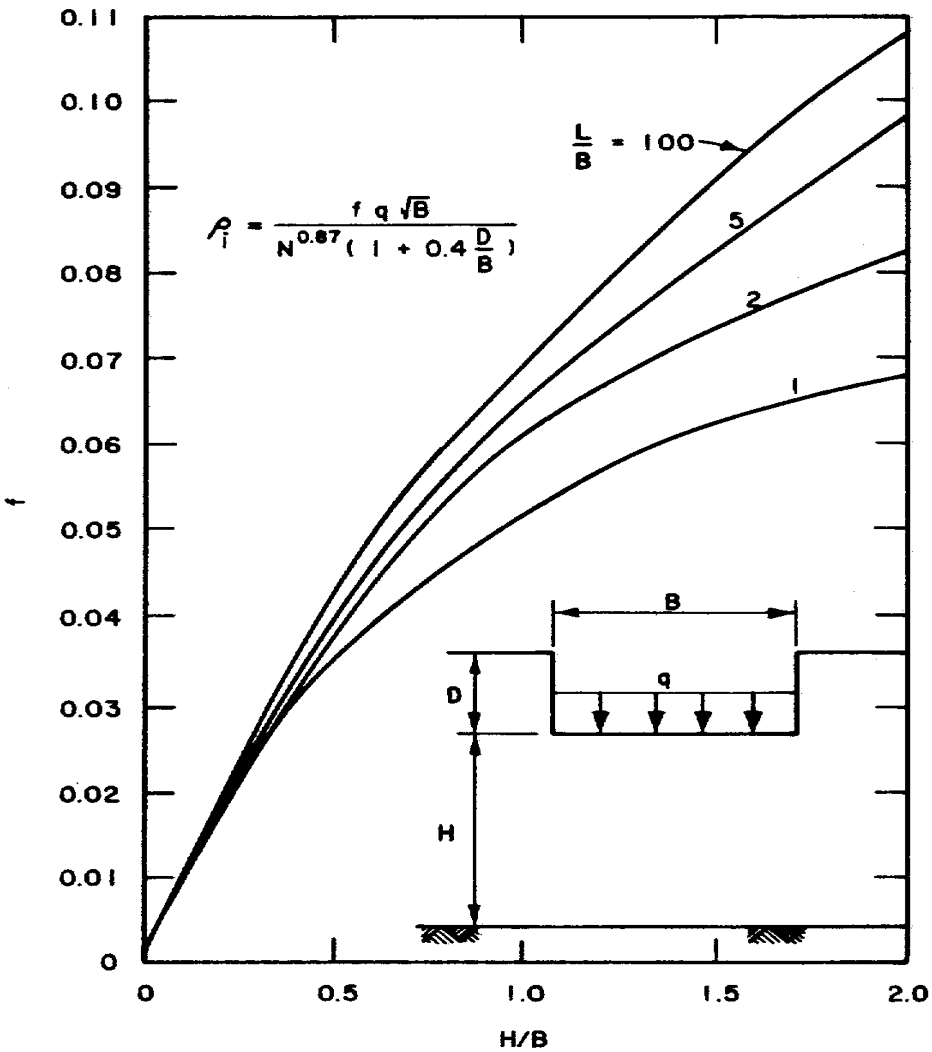


Figure 3-2. Settlement from the standard penetration test (Data from item 60)

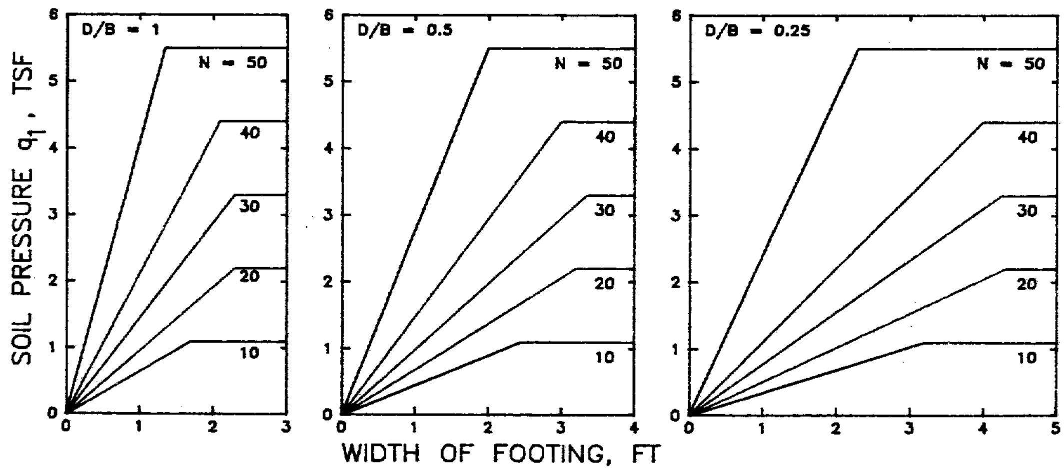
Equation 3-5 calculates settlements 2/3 of the Terzaghi and Peck method (item 51) as recommended by Peck and Bazarra (item 50).

(1) Water table correction. The correction Cw is given by

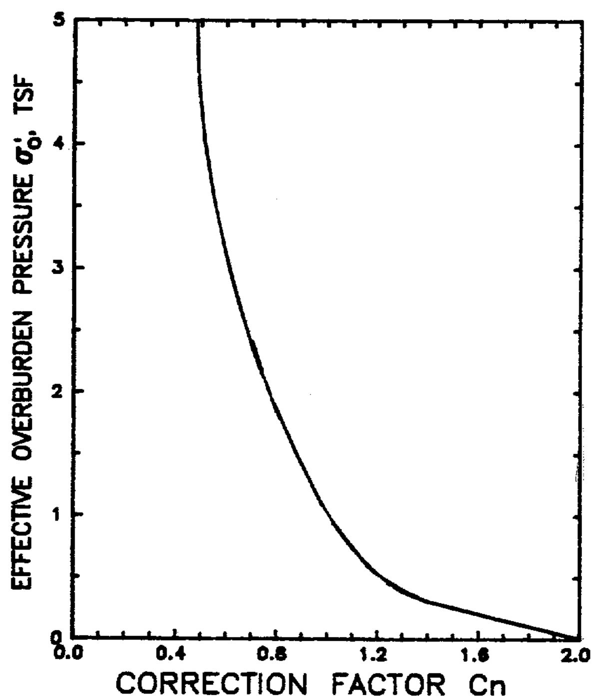
$$Cw = 0.5 + 0.5 \bullet \frac{Dw}{D + B} \tag{3-7}$$

where Dw = depth to groundwater level, ft. The correction factor Cw = 0.5 for a groundwater level at the ground surface. The correction factor is 1 if the sand is dry or if the groundwater level exceeds the depth D + B below the ground surface.

(2) Overburden pressure correction. The correction factor Cn is found from Figure 3-3b as a function of the effective vertical overburden pressure σ_o' .



a. EVALUATION OF SOIL PRESSURE q_1 FROM CORRECTED BLOWCOUNT N′ AND EMBEDMENT DEPTH/FOOTING WIDTH RATIO D/B



b. CORRECTION Cn FOR EFFECTIVE OVERBURDEN PRESSURE σ_o'

Figure 3-3. Charts for Modified Terzaghi and Peck Approximation. Reprinted by permission of John Wiley & Sons, Inc. from Foundation Engineering, 2nd Edition, Copyright © 1974 by R. B. Peck, W. E. Hanson, and T. H. Thornburn, pp 309, 312

d. Schmertmann Approximation. This procedure provides settlement compatible with field measurements in many different areas. The analysis assumes that the distribution of vertical strain is compatible with a linear elastic half space subjected to a uniform pressure (item 55)

$$\rho_i = C_1 \bullet C_t \bullet \Delta p \bullet \sum_{i=1}^{n} \frac{\Delta z_1}{E_{si}} \bullet Izi \tag{3-8}$$

where

C_1 = correction to account for strain relief from embedment, $1 - 0.5\sigma'_{od}/\Delta p \geq 0.5$
σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf
Δp = net applied footing pressure, $q - \sigma'_{od}$, tsf
C = correction for time dependent increase in settlement, $1 + 0.2 \cdot \log_{10}(t/0.1)$
t = time, years
E_{si} = elastic modulus of soil layer i, tsf
Δz_i = depth increment i, 0.2B, ft
Izi = influence factor of soil layer i, Figure 3-4

Settlement may be calculated with the assistance of the calculation sheet, Figure 3-5. The time-dependent increase in settlement is related with creep and secondary compression as observed in clays.

(1) Influence factor. The influence factor Iz is based on approximations of strain distributions for square or axisymmetric footings and for infinitely long or plane strain footings observed in cohesionless soil, which are similar to an elastic medium such as the Boussinesq distribution, Figure 1-2. The peak value of the influence factor Izp in Figure 3-4 is (item 59)

$$Izp = 0.5 + 0.1 \left[\frac{\Delta p}{\sigma'_{Izp}}\right]^{1/2} \tag{3-9a}$$

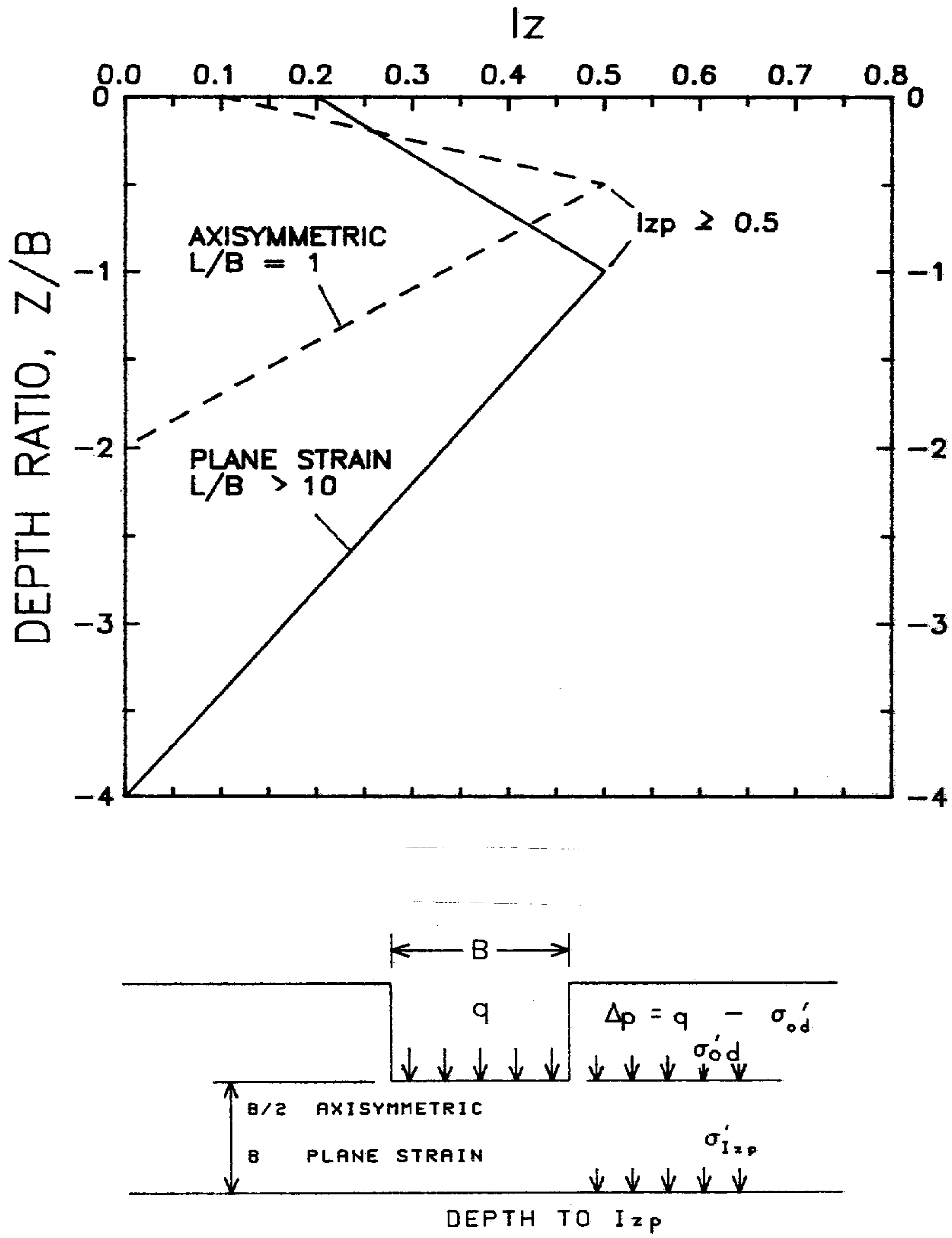
$$\textit{Axisymmetric: } L/B = 1 \qquad \sigma'_{Izp} = 0.5 \bullet B \bullet \gamma' + D \bullet \gamma' \tag{3-9b}$$

$$\textit{Plane Strain: } L/B \geq 10 \qquad \sigma'_{Izp} = B \bullet \gamma' + D \bullet \gamma' \tag{3-9c}$$

where

σ'_{Izp} = effective overburden pressure at the depth of Izp, tsf
γ' = effective unit weight (wet soil unit weight γ less unit weight of water) in units of ton/cubic foot.
D = excavated or embeded depth, ft

The parameter σ_{Izp} may be assumed to vary linearly between Equations 3-9b and 3-9c for L/B between 1 and 10. Iz may be assumed to vary linearly between 0.1 and 0.2 on the Iz axis at the ground surface for L/B between 1 and 10 and Z/B may be assumed to vary linearly between 2 and 4 on the Z/B axis for L/B between 1 and 10.



Z = DEPTH BELOW FOOTING BOTTOM, FT
B = FOOTING WIDTH, FT
Iz = DEPTH INFLUENCE FACTOR
Izp = PEAK DEPTH INFLUENCE FACTOR

Figure 3-4. Recommended strain influence factors for Schmertmann's Approximation. Reprinted with permission of the American Society of Civil Engineers from the Journal of the Geotechnical Engineering Division, Vol 104, 1978, "Improved Strain Influence Factor diagram", by J. M. Schmertmann, J. P. Hartman, and P. R. Brown, p 1134

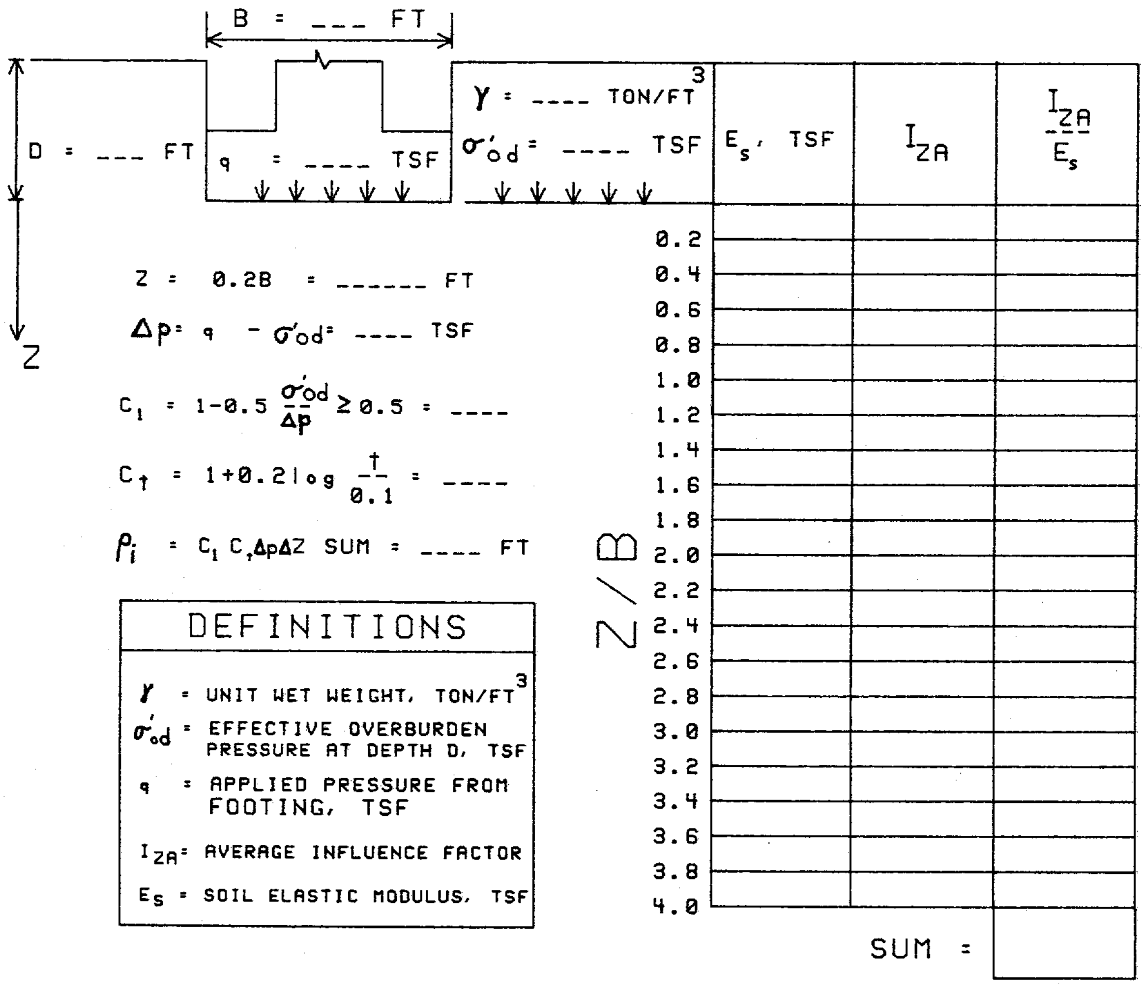


Figure 3-5. Settlement calculation sheet for cohesionless soil using Schmertmann's method

(2) Elastic modulus. Elastic modulus E_{si} may be estimated from results of the mechanical (Dutch Static) Cone Penetration Test (CPT) (item 59)

$$\textit{Axisymmetric Footings: } L/B = 1 \qquad E_{si} = 2.5 \cdot q_c \tag{3-10a}$$

$$\textit{Plane Strain Footings: } L/B \geq 10 \qquad E_{si} = 3.5 \cdot q_c \tag{3-10b}$$

where q_c is the cone tip bearing resistance in units of tsf. E_{si} may be assumed to vary linearly between Equations 3-10a and 3-10b for L/B between 1 and 10 . SPT data may also be converted to Dutch cone bearing capacity by the correlations in Table 3-1. The estimated average elastic modulus of each depth increment may be plotted in the E_s column of Figure 3-5.

Table 3-1

Correlations Between Dutch Cone Tip Resistance q_c and Blow Count N from the SPT (Data from Item 55)

Soil	q_c/N*
Silts, sandy silts, slightly cohesive silt-sand	2
Clean, fine to medium sands and slightly silty sands	3.5
Coarse sands and sands with little gravel	5
Sandy gravel and gravel	6

*Units of q_c are in tsf and N in blows/ft

(3) Calculation of settlement. Iz/E_s is computed for each depth increment z/B and added to obtain SUM, Figure 3-5. Immediate settlement of the soil profile may then be calculated as shown on Figure 3-5. If a rigid base lies within $z = 2B$, then settlement may be calculated as shown down to the rigid base.

e. Burland and Burbidge Approximation. This procedure based on 200 SPT case studies predicts settlements less than most of these methods (item 4).

(1) Immediate settlement of sand and gravel deposits may be estimated by

$$\Delta P'_{ave} > \sigma'_p: \qquad \rho_i = f_s \bullet f_i \bullet \left[\left(\Delta P'_{ave} - \frac{2}{3}\sigma'_p \right) \bullet B^{0.7} \bullet I_c \right] \qquad (3\text{-}11a)$$

$$\Delta P'_{ave} < \sigma'_p: \qquad \rho_i = f_s \bullet f_i \bullet \Delta P'_{ave} \bullet \frac{I_c}{3} \qquad (3\text{-}11b)$$

where

f_s = shape correction factor, $[(1.25 \cdot L/B)/(L/B + 0.25)]^2$
f_1 = layer thickness correction factor, $H/z_1 \cdot (2 - H/z_1)$
$\Delta P'_{ave}$ = average effective bearing pressure, $q_{oave} + \sigma'_{oave}$, tsf
q_{oave} = average pressure in stratum from foundation load, tsf
σ'_{oave} = average effective overburden pressure in stratum H, tsf
σ'_p = maximum effective past pressure, tsf
H = thickness of layer, ft
z_1 = depth of influence of loaded area, ft
I_c = compressibility influence factor, $\approx 0.23/(N_{ave}^{1.4})$ with coefficient of correlation 0.848
N_{ave} = average SPT blowcount over depth influenced by loaded area

(a) The depth of influence z_1 is taken as the depth at which the settlement is 25 percent of the surface settlement. This depth in feet may be approximated by $1.35B^{0.75}$ where N_{ave} increases or is constant with depth. z_i is taken as 2B where N_{ave} shows a consistent decrease with depth.

(b) N_{ave} is the arithmetic mean of the measured N values within the depth of influence z_1 . N_{ave} is not corrected for effective overburden pressure, but instead considers compressibility using I_c . The arithmetic mean of the measured N_{ave} should be corrected to $15 + 0.5(N_{ave} - 15)$ when $N_{ave} > 15$ for very fine and silty sand below the water table and multiplied by 1.25 for gravel or sandy gravel.

(c) The probable limits of accuracy of Equations 3-11 are within upper and lower bound values of I_c given by

$$0.08/(N_{ave})^{1.3} \leq I_c \leq 1.34/N_{ave})^{1.67} \tag{3-12}$$

(2) Settlement after time t at least 3 years following construction from creep and secondary compression effects may be estimated by

$$\rho_t = f_t \bullet \rho_i \tag{3-13}$$

where

$f_t = 1 + R_3 + R_t \cdot \log t/3$
R_3 = time-dependent settlement ratio as a proportion of ρ_i during first 3 years following construction, ≈ 0.3
R_t = time-dependent settlement ratio as a proportion of ρ_i for each log cycle of time after 3 years, ≈ 0.2

Values of R_3 and R_t are conservative based on 9 case records (item 4).

f. Dilatometer Approximation. The dilatometer consists of a stainless steel blade 96 mm wide and 15 mm thick with a sharp edge containing a stainless steel membrane centered and flush with one side of the blade. The blade is preferably pushed (or driven if necessary) into the soil. A pressure-vacuum system is used to inflate/deflate the membrane a maximum movement of 1.1 mm against the adjacent soil (item 58).

(1) Calculation. This procedure predicts settlement from evaluation of one-dimensional vertical compression or constrained modulus E_d by the DMT

$$\rho_i = \frac{q_{oave} \bullet H}{E_d} \tag{3-14}$$

where

q_{oave} = average increase in stress caused by the applied load, tsf
H = thickness of stratum at depth z where q_{oave} is applicable, ft
E_d = constrained modulus, $R_D E_s$, tsf
R_D = $(1 - \nu_s)/[(1 + \nu_s)(1 - 2\nu_s)]$, factor that varies from 1 to 3 relates E_d to Young's soil modulus E_s
ν_s = Poisson's ratio

Refer to Appendix D for additional information on elastic parameters. The influence of prestress on settlement may be corrected using results of DMT and CPT tests after Schmertmann's approximation (item 37) to reduce settlement overestimates.

(2) Evaluation of elastic modulus. The dilatometer modulus of soil at the depth of the probe is evaluated as 34.7 times the difference in pressure between the deflated and inflated positions of the membrane. Young's elastic modulus has been found to vary from 0.4 to 10 times the dilatometer modulus (item 39). A Young's elastic modulus equal to the dilatometer modulus may be assumed for many practical applications in sands.

(3) Adjustment for other soil. The constrained modulus E_d may be adjusted for effective vertical stress σ'_o other than that of the DMT for overconsolidated soil and normally consolidated clay by

$$E_d = m \bullet \sigma'_o \tag{3-15a}$$

where

$m = [(1 + e)/C_c] \cdot \ln 10$

e = void ratio

C_c = compression index

The constrained modulus for normally consolidated silts and sand is

$$E_d = m \bullet (\sigma'_o)^{0.5} \tag{3-15b}$$

where σ'_o is the effective vertical overburden pressure, tsf. These settlements include time-dependent settlements excluding secondary compression and creep. Total settlement of a heterogeneous soil with variable E_d may be estimated by summing increments of settlement using Equation 3-14 for layers of thickness H.

3-4. Recommendations. A minimum of three methods should be applied to estimate a range of settlement. Settlement estimates based on in situ test results are based on correlations obtained from past experience and observation and may not be reliable.

a. Evaluation from SPT Data. The Alpan (Equation 3-2), Schultze and Sherif (Equation 3-4), Modified Terzaghi and Peck (Equation 3-5) approximations should all be applied to estimate immediate settlement if blowcount data from SPT are available. The Burland and Burbidge approximation (Equations 3-11) should be applied if the maximum past pressure of the soil can be estimated; this approximation using Equation 3-12 may also be applied to estimate a range of settlement.

b. Evaluation from CPT Data. The Schmertmann approximation (Equation 3-8) should be used to estimate settlement if CPT data are available.

c. Evaluation from DMT Data. The Dilatometer approximation (Equation 3-14) should be used if data from this test are available. The range of settlement may be determined by assuming minimum and maximum values of the factor R_D of 1 and 3.

d. Evaluation from PMT Data. The pressuremeter unload-reload modulus from the corrected pressure versus volume change curve is a measure of twice the shear modulus, Appendix D-2d. The Young's elastic modulus may be evaluated from the shear modulus, Table D-2, and settlement estimated from Equation 3-8. The constrained modulus may be evaluated from Young's elastic modulus, Table D-2, and settlement estimated from Equation 3-14.

e. Long-Term Settlement. The Schmertmann and Burland and Burbidge approximations may be used to estimate long-term settlement in cohesionless soil from CPT and SPT data. The constrained modulus E_d may also be adjusted to consider consolidation from Equations 3-15 and settlement estimated from Equation 3-14. Refer to items 39 and 58 for detailed information on evaluation of the constrained modulus.

3-5. Application. A footing 10-ft square is to be constructed 3 ft below grade on medium dense ($\gamma = \gamma' = 0.06$ ton/ft^3) and moist sand with total stratum thickness of 13 ft (H = 10'). The water table is at least 5 ft below the base of the footing. The effective vertical overburden pressure at the bottom of the footing is $\sigma'_{od} = \gamma' \cdot z = 0.06 \cdot 3 = 0.18$ tsf . The bearing pressure of the footing on the sand $q = 2$ tsf . Field data indicate an average blowcount in the sand $N_{ave} = 20$ blows/ft and the cone tip bearing resistance is about 70 tsf. The average elastic modulus determined from dilatometer and pressuremeter tests indicated $E_s = 175$ tsf. Refer to Figure 3-6 for a schematic description of this problem. Estimates of settlement of this footing at end of construction (EOC) and 10 years after construction are required.

(1) Results of the settlement computations comparing several of the above methods are shown in Table 3-2.

(a) Figure 3-6 illustrates computation of settlement by Schmertmann's method.

(b) Computation of settlement by the Burland and Burbidge and dilatometer approximations requires an estimate of the average effective bearing pressure $\Delta P'_{ave}$. Assuming that the 2:1 stress distribution of Figure C-1 is adequate, the average pressure from the foundation load is

$$q_{oave} = \frac{q + \Delta\sigma_z}{2}$$

where $\Delta\sigma_z$ is found from Equation C-2. Therefore, if B = L = H = 10 ft and $Q = q \cdot B \cdot L$, then

$$q_{oave} = 0.5 \left[2.0 + \frac{2.0 \bullet 10^2}{(10 + 10)^2} \right] = 1.25 \ tsf$$

The average effective overburden pressure $\sigma'_{oave} = 0.06 \cdot (3 + 13)/2.0$ or 0.48 tsf. The average effective bearing pressure $\Delta P'_{ave}$ is therefore 1.25 + 0.48 = 1.73 tsf. The soil is assumed normally consolidated; therefore, $\sigma'_p = \sigma'_{od} = 0.18$ tsf and Equation 3-11a is applicable. Factor $f_s = 1.0$, $H/z_1 = 1.31$ and $f_1 = 0.91$. $I_c = 0.23/(20)^{1.4} = 0.0035$.

Table 3-2

Estimation of Immediate Settlement for Example Application of Footing on Cohesionless Soil

a. Calculations

Method	Equation	Calculations
		$B = 10\ ft,\ \frac{L}{B} = 1\ ,\ m' = 1.0\ ,\ N' = 65\ blows/ft$
Alpan (item 1)		$N = 20\ blows/ft$ and $\sigma'_o = 0.18\ tsf$, Figure 3-1a
		$\alpha_o = 0.05\ in/tsf$, Figure 3-1b
	3-2	$\rho_i = 1 \bullet \left[\frac{2 \bullet 10}{1 + 10}\right]^2 \bullet \frac{0.05}{12} \bullet 2 = 0.027\ ft$ or $0.33\ inch$
Schultz and Sherif (item 60)		$\frac{H}{B} = 1,\ \frac{D}{B} = 0.3\ ,\ f = 0.052$, Figure 3-2
	3-4	$\rho_i = \frac{0.052 \bullet 2 \bullet \sqrt{10}}{20^{0.87}(1 + 0.4 \bullet 0.3)} = 0.022\ ft$ or $0.33\ inch$
Modified Terzaghi and Peck (item 51)		$C_n = 1.6$, Figure 3-3b
		$C_w = 1$
	3-6	$N' = 1 \bullet 1.6 \bullet 20 = 32$
		$q_1 = 3.5\ tsf$, Figure 3-3a
	3-5	$\rho_i = \frac{1}{18} \bullet \frac{2}{3.5} = 0.031\ ft$ or $0.38\ inch$ ($0.57\ inch$ ignoring Peck and Bazarra 1/3 reduction)
Schmertmann (item 55)		Refer to Figure 3-6
		$q_c = 70\ tsf$,
	3-10a	$E_{si} = 2.5 \bullet 70 = 175\ tsf$,
	3-9b	$\sigma'_{Izp} = 0.5 \bullet 10 \bullet 0.06 + 3 \bullet 0.6 = 0.48\ tsf$
	3-9a	$Izp = 0.5 + 0.1 \left[\frac{1.82}{0.48}\right]^{0.5} = 0.695$

z, FT	z/B	Iz	Iza
0	0.0	0.10	
			0.22
2	0.2	0.34	
			0.46
4	0.4	0.58	
			0.62
6	0.6	0.65	
			0.61
8	0.8	0.56	
			0.51
10	1.0	0.46	

3-8

ρ_i at EOC $= 0.95 \bullet 1.0 \bullet 1.82 \bullet 2.0 \bullet 0.0138 = 0.0477\ ft$ or $0.57\ inch$

ρ_i after 10 years $= 0.0477 \bullet 1.4 = 0.0668\ ft$ or $0.80\ inch$

Table 3-2. Concluded

Method	Equation	Calculations
Burland and Burbidge (item 4)	3-11a	*End of* $\rho_i = 1 \bullet 0.91[(1.73 - 0.67 \bullet 0.18) \bullet 10^{0.7} \bullet 0.0035]$
		Construction: $= 0.028$ *ft or* 0.34 *inch*
		After 10 *years*: $f_t = 1 + 0.3 + 0.2\log\frac{10}{3} = 1.4$
		$\rho_t = 0.028 \bullet 1.4 = 0.039$ *or* 0.47 *inch*
	3-13	$I_{cmin} \approx 0.08/(20)^{1.3} = 0.0016$
	3-12	$\rho_{imin} = 0.028 \bullet 0.0016/0.0035 = 0.013$ *ft* *or* 0.15 *inch*
	3-12	$I_{cmax} \approx 1.34/(20)^{1.67} = 0.009$
		$\rho_{imax} = 0.028 \bullet 0.009/0.0035 = 0.073$ *ft* *or* 0.87 *inch*
Dilatometer (item 58)		$q_{oave} = 1.25$ *tsf* , $H = 10$ *ft* , $E_s = 175$ *tsf* , ν_s *not needed*
		Minimum Settlement: $R_D = 3$, $E_d = 3 \bullet 175 = 525$ *tsf*
	3-14	$\rho_{imin} = 1.25 \bullet 10/525 = 0.024$ *ft or* 0.29 *inch*
		Maximum Settlement: $R_D = 1$, $E_d = 1 \bullet 175 = 175$ *tsf*
	3-14	$\rho_{imax} = 1.25 \bullet 10/175 = 0.071$ *ft or* 0.86 *inch*

b. Comparison

Method	Immediate Settlement, ft (in.)	
Alpan	0.027	(0.33)
Schultz and Sherif	0.022	(0.27)
Modified Terzaghi and Peck	0.031	(0.38)
Schmertmann	0.048	(0.57)
Burland and Burbidge	0.028	(0.34)
Dilatometer	0.024 - 0.071	(0.29-0.86)

(2) A comparison of results in Table 3-2b shows that the Alpan, Schultze and Sherif, Modified Terzaghi and Peck, and Burland and Burbidge methods provide consistent settlements of about 0.3 to 0.4 inch. The Schmertmann method is reasonably conservative with settlement of 0.57 inch. This settlement is the same as that from the Modified Terzaghi and Peck method ignoring the 1/3 reduction recommended by Peck and Bazarra (item 50). Long-term settlement is 0.5 (Burland and Burbidge) and 0.8 inch (Schmertmann) after 10 years. The expected range of settlement is 0.2 to 1.0 inch after the Burland and Burbidge method and 0.3 to 0.9 inch from the dilatometer. Settlement is not expected to exceed 1 inch.

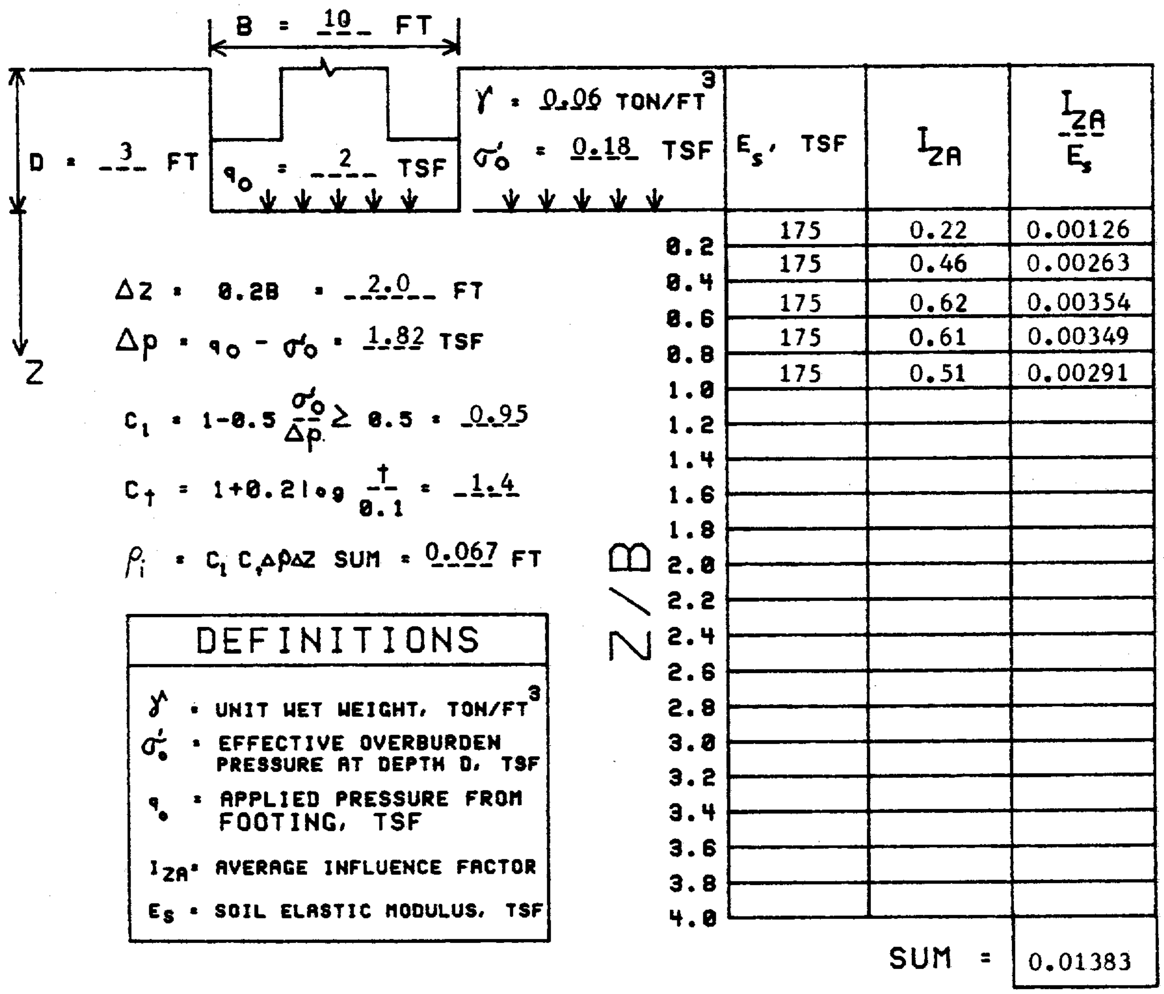


Figure 3-6. Estimation of immediate settlement by Schmertmann's method

Section II. Immediate Settlement of Cohesive Soil for Static Loads

3-6. General. Static loads cause immediate and long-term consolidation settlements in cohesive or compressible soil. The stress in the soil caused by applied loads should be estimated (paragraph 1-5d) and compared with estimates of the maximum past pressure (paragraph 1-5a). If the stress in the soil exceeds the maximum past pressure, then primary consolidation and secondary compression settlement may be significant and should be evaluated by the methods in Sections III and IV. Immediate rebound or heave may occur in compressible soil at the bottom of excavations, but may not be a design or construction problem unless rebound causes the elevation of the basement or first floor to exceed specifications or impair performance.

3-7. Rebound in Excavations. Most rebound in excavations lying above compressible strata occurs from undrained elastic unloading strains in these strata. Additional long-term heave due to wetting of the soil following reduction in pore water pressure following removal of overburden in excavations is discussed in Section I, Chapter 5. Rebound of compressible soil in excavations may be approximated as linear elastic by (item 2)

$$S_{RE} = F_{RD} \bullet F_{RS} \bullet \frac{\gamma D^2}{E_s^*} \tag{3-16}$$

where

S_{RE} = undrained elastic rebound, ft
F_{RD} = rebound depth factor, Figure 3-7a
F_{RS} = rebound shape factor, Figure 3-7b
γ = wet unit weight of excavated soil, tons/ft^3
D = depth of excavation, ft
E_s^* = equivalent elastic modulus of soil beneath the excavation, tsf

The equivalent elastic modulus E_s^* may be estimated by methods described in Appendix D, Elastic Parameters. The compressible stratum of depth H is assumed to be supported on a rigid base such as unweathered clay shale, rock, dense sand or gravel. An example application is provided in Figure 3-7c.

3-8. Immediate Settlement in Cohesive Soil. The immediate settlement of a structure on cohesive soil (see paragraph 1-5c for definition) consists of elastic distortion associated with a change in shape without volume change and, in unsaturated clay, settlement from a decrease in volume. The theory of elasticity is generally applicable to cohesive soil.

a. Improved Janbu Approximation. The average immediate settlement of a foundation on an elastic soil may be given by (item 9)

$$\rho_i = \mu_o \bullet \mu_1 \bullet \frac{q \bullet B}{E_s^*} \tag{3-17}$$

where

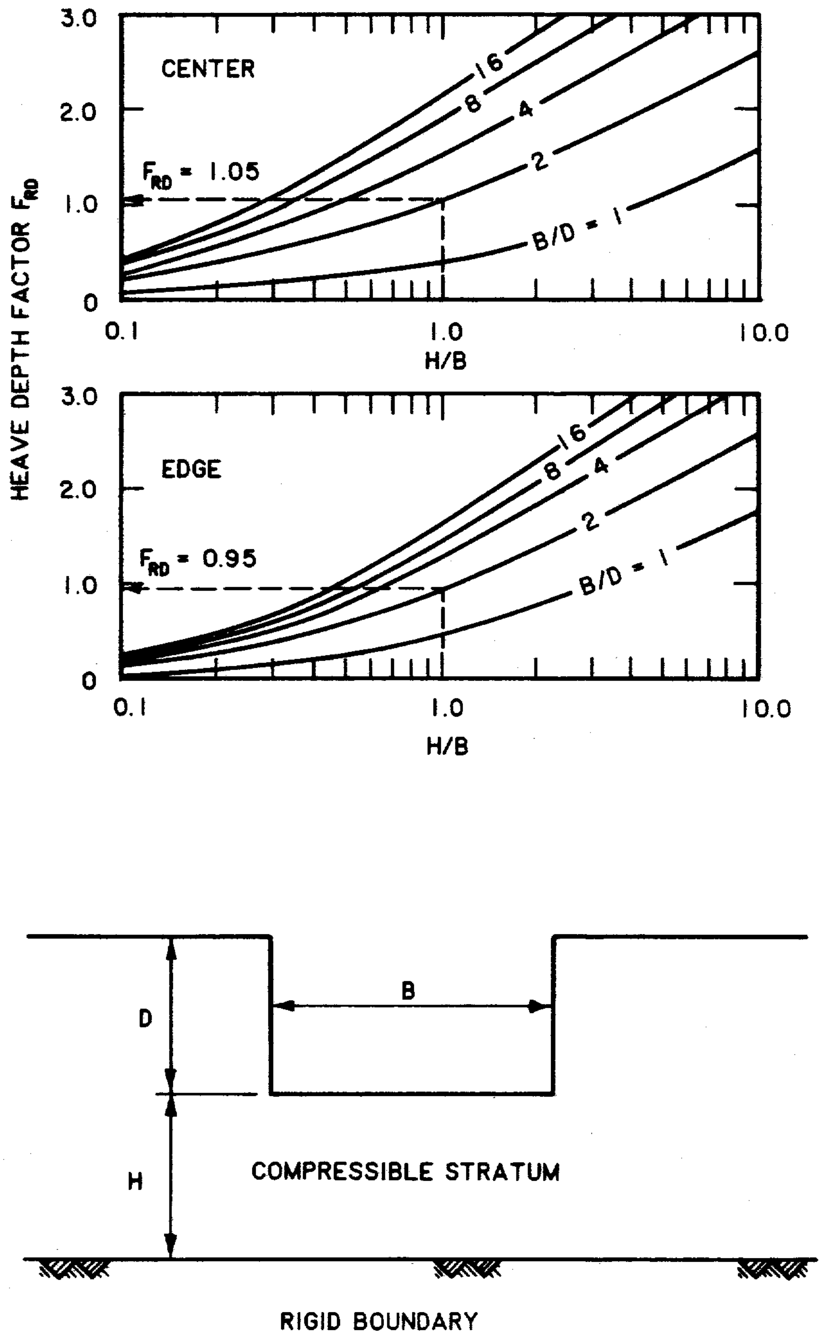
μ_o = influence factor for depth D of foundation below ground surface, Figure 3-8
μ_1 = influence factor for foundation shape, Figure 3-8
E_s^* = equivalent Young's modulus of the soil, tsf

(1) A comparison of test calculations and results of finite element analysis have indicated errors from Equation 3-17 usually less than 10 percent and always less than 20 percent for H/B between 0.3 and 10 , L/B between 1 and 5 , and D/B between 0.3 and 3 , Figure 3-8. Reasonable results are given in most cases when μ_o is set equal to unity. Poisson's ratio ν_s is taken as 0.5 .

(2) E_s^* may be estimated by methods in Appendix D.

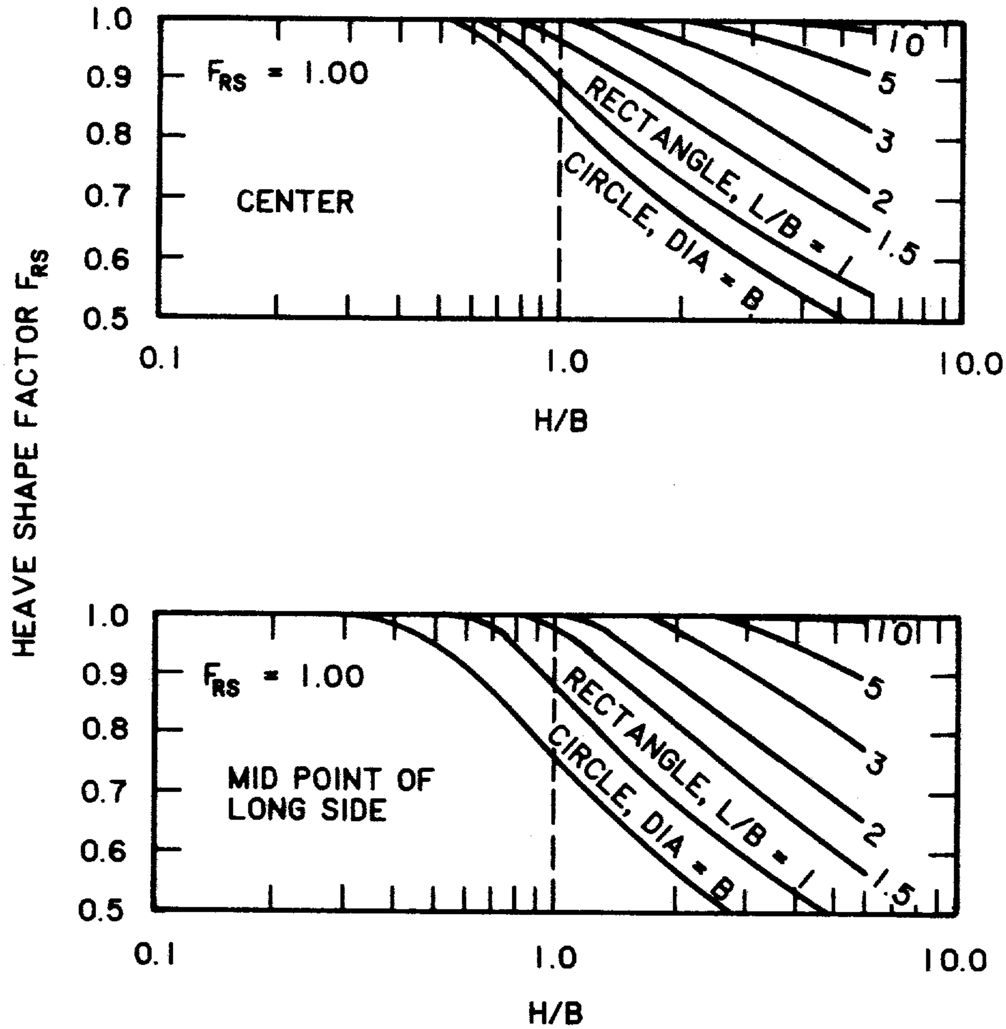
b. Perloff Approximation. The immediate vertical settlement beneath the center and edge of a mat or footing may be given by (item 52)

$$\rho_i = I \bullet q \bullet B \bullet \left[\frac{1 - \nu_s^2}{E_s}\right] \bullet \alpha \tag{3-18}$$



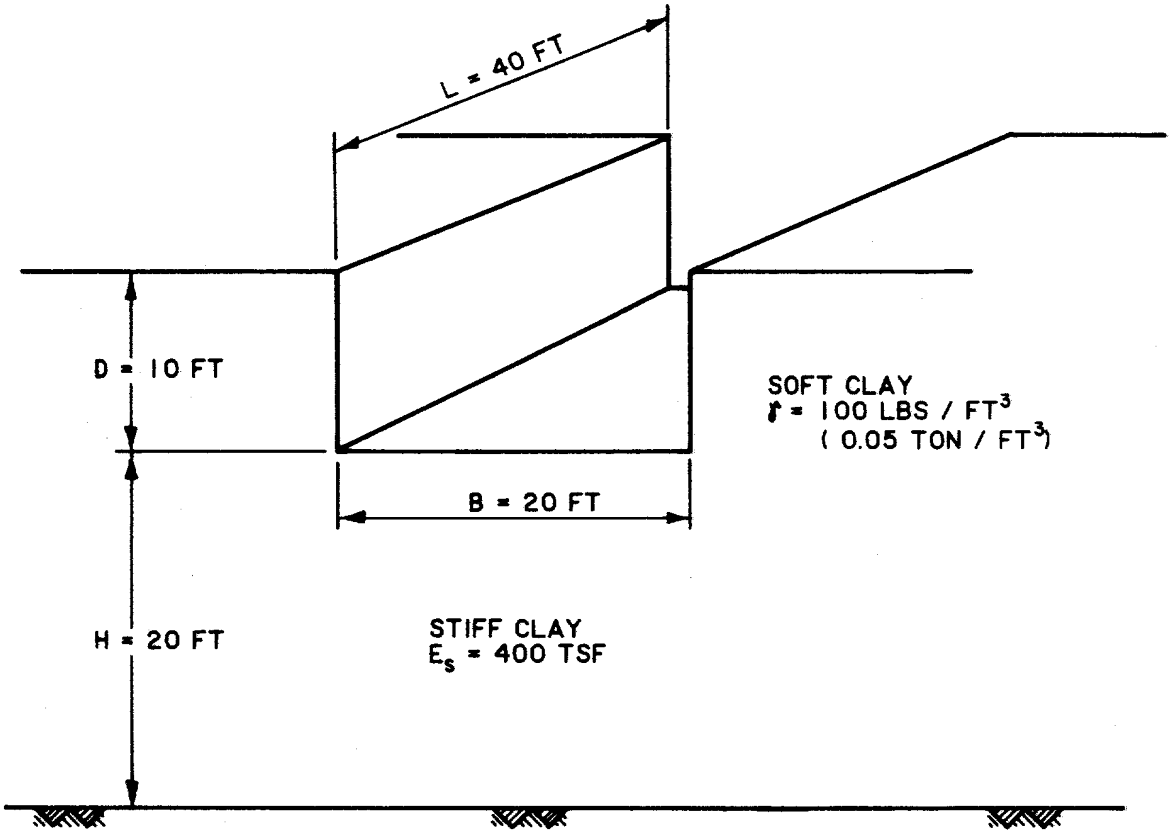
a. REBOUND DEPTH FACTOR F_{RD}

Figure 3-7. Factors to calculate elastic rebound in excavations. Reprinted by permission of the author G. Y. Baladi from "Distribution of Stresses and Displacements Within and Under Long Elastic and Viscoelastic Embankments," Ph.D. Thesis, 1968, Purdue University



b. REBOUND SHAPE FACTOR F_{RS}

Figure 3-7. (Continued)



$$S_{RE} = F_{RD} F_{RS} \frac{\gamma D^2}{E_s}$$

$$\frac{H}{B} = \frac{20}{20} = 1.0$$

$$\frac{B}{D} = \frac{20}{10} = 2.0$$

$$\frac{L}{B} = \frac{40}{20} = 2.0$$

FROM a: $F_{RD_{CENTER}} = 1.05$

$F_{RD_{EDGE}} = 0.95$

FROM b: $F_{RS_{CENTER}} = 1.00$

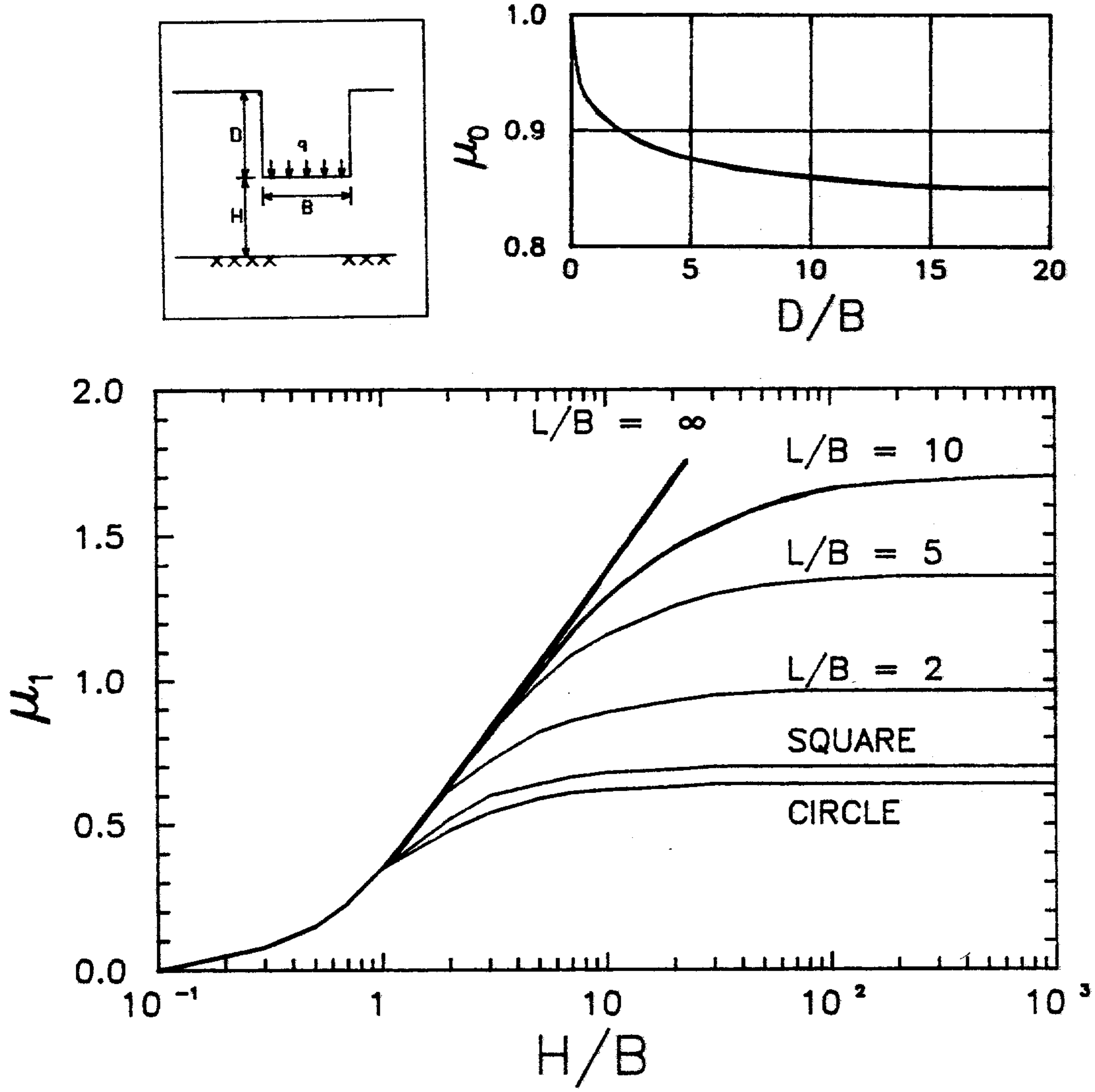
$F_{RS_{MID\ EDGE}} = 1.00$

$$S_{RE_{CENTER}} = \frac{(1.05)(1.00)(0.05)(10)^2}{400} = 0.013 \text{ FT OR } 0.16 \text{ INCH}$$

$$S_{RE_{MID\ EDGE}} = \frac{(0.95)(1.00)(0.05)(10)^2}{400} = 0.012 \text{ FT OR } 0.14 \text{ INCH}$$

c. **EXAMPLE CALCULATION**

Figure 3-7. (Concluded)



$$\rho_i = \mu_o \cdot \mu_1 \cdot \frac{q \cdot B}{E_s^*}$$

L = LENGTH, FT

B = WIDTH, FT

E_s^* = EQUIVALENT YOUNG'S SOIL MODULUS, TSF

ν_s = POISSON'S RATIO, 0.5

Figure 3-8. Chart for estimating immediate settlement in cohesive soil. Reprinted by permission of the National Research Council of Canada from Canadian Geotechnical Journal, Vol 15, 1978, "Janbu, Bjerrum, and Kjaernsli's Chart Reinterpreted", by J. T. Christian and W. D. Carrier III, p 127.

where

I = influence factor for infinitely deep and homogeneous soil, Table 3-3a
E_s = elastic soil modulus, tsf
ν_s = soil Poisson's ratio
α = correction factor for subgrade soil, Table 3-3b

The influence factor I may be modified to account for heterogeneous or multilayered soil usually encountered in practice. If the upper soil is relatively compressible and underlain by stiff clay, shale, rock, or dense soil, then the compressible soil stratum may be approximated by a finite layer of depth H supported on a rigid base. The influence factor I is given in Figure 3-9 for settlement beneath the center and midpoint of the edge of flexible foundations. If the subgrade soil supporting the foundation with modulus E_{s1} and thickness H is underlain by less rigid infinitely deep material with modulus E_{s2} , then settlement at the center of a uniformly loaded circular area placed on the surface of the more rigid soil is corrected with the factor α , Table 3-3b.

c. Kay and Cavagnaro Approximation. The immediate elastic settlement at the center and edge of circular foundations and foundations with length to width ratios less than two may be evaluated for layered elastic soil by the graphical procedure, Figure 3-10 (item 31). The method considers the relative rigidity of the foundation relative to the soil and can evaluate the differential displacement between the center and edge of the foundation.

3-9. Recommendations.

a. Janbu Approximation. The Janbu approximation is recommended when an average computation of settlement is required for a wide range of depths, lengths, and widths of foundations supported on compressible soil of depth H .

b. Perloff Approximation. The Perloff approximation should be used when total and differential settlement is required beneath flexible foundations located at or near the surface of the soil; settlements may be evaluated at the center, corner, and middle edges of both the short and long sides of the foundation.

c. Kay and Cavagnaro Approximation. The Kay and Cavagnaro approximation should be used when total and differential settlement is required beneath footings and mats of a given stiffness supported on compressible soil of variable elastic modulus; settlement may be evaluated at the center and edge for a given foundation depth. A reasonable estimate of Poisson's ratio for cohesive soil is 0.4, Appendix D-4.

d. Linear Modulus Increase. The Gibson model described in Appendix D-2d may be used if the elastic modulus may be assumed zero at the ground sur-face. A parametric analysis using the Kay and Cavagnaro graphical procedure for an elastic modulus increasing linearly with depth indicates that the center settlement beneath a foundation may be calculated by

$$\rho_c = \frac{q}{k}\left[0.7 + (2.3 - 4\nu_s)\log_{10} n\right] \tag{3-19}$$

Table 3-3

Factors for Estimating Immediate Settlement in Cohesive Soil

a. Shape and Rigidity Factor I for Calculating Settlements of Points on Loaded Areas at the Surface of an Elastic Half-Space (Data from Item 52)

Shape	Center Center	Corner Corner	Middle Short Side	Middle Long Side
Circle	1.00	0.64	0.64	0.64
Rigid circle	0.79			
Square	1.12	0.56	0.76	0.76
Rigid square	0.99			
Rectangle Length/Width				
1.5	1.36	0.67	0.89	0.97
2	1.52	0.76	0.98	1.12
3	1.78	0.88	1.11	1.35
5	2.10	1.05	1.27	1.68
10	2.53	1.26	1.49	2.12
100	4.00	2.00	2.20	3.60
1000	5.47	2.75	2.94	5.03
10000	6.90	3.50	3.70	6.50

b. Correction Factor α at the Center of a Circular Uniformly Loaded Area of Width B on an Elastic Layer of Modulus E_{s1} of Depth H Underlain by a Less Stiff Elastic Material of Modulus E_{s2} of Infinite Depth

	E_{s1}/E_{s2}				
H/B	1	2	5	10	100
0	1.000	1.000	1.000	1.000	1.000
0.1	1.000	0.972	0.943	0.923	0.760
0.25	1.000	0.885	0.779	0.699	0.431
0.5	1.000	0.747	0.566	0.463	0.228
1.0	1.000	0.627	0.399	0.287	0.121
2.5	1.000	0.550	0.274	0.175	0.058
5	1.000	0.525	0.238	0.136	0.036
∞	1.000	0.500	0.200	0.100	0.010

Reprinted from D. M. Burmister 1965, "Influence Diagrams for Stresses and Displacements in a Two-Layer Pavement System for Airfields", Contract NBY 13009, Department of the Navy, Washington, D. C. (item 7)

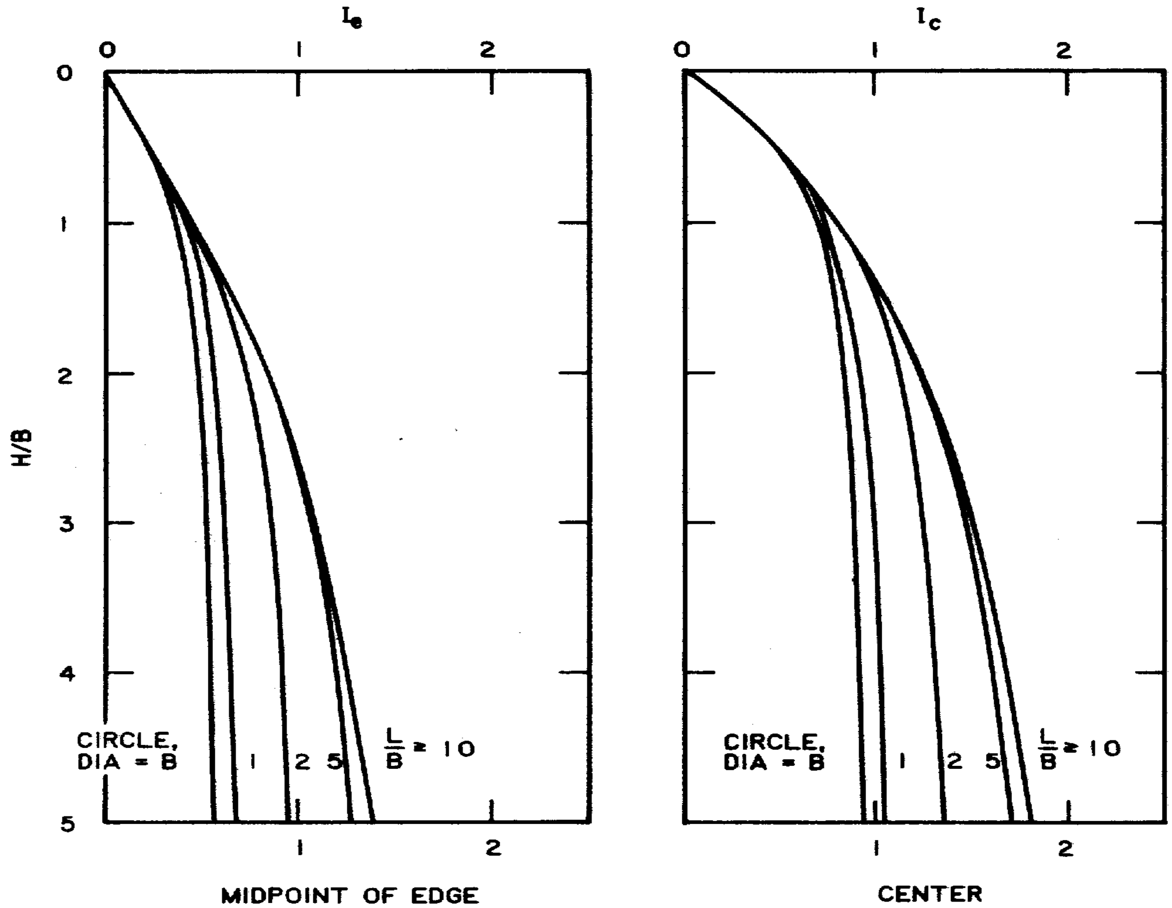


Figure 3-9. Influence factor I for settlement of a completely flexible mat or footing of width B and length L on a finite elastic material of depth H supported on a rigid base. Data taken with permission of McGraw-Hill Book Company from Tables 2-4 and 2-5, Foundations of Theoretical Soil Mechanics, by M. E. Harr, 1966, p 98, 99.

where

n = $kR/(E_o + kD_b)$
k = constant relating the elastic modulus with depth; i.e., $E_o = E_s + kz$, ksf/ft
R = equivalent radius of the mat or footing, $\sqrt{LB/\pi}$
E_o = elastic soil modulus at the ground surface, ksf
D_b = depth of the mat base or stiffening beams beneath the ground surface, ft

Edge and corner settlement of a flexible mat or footing will be approximately 1/2 and 1/4 of the center settlement, respectively. Differential movement of the mat or footing may be calculated from Figure 3-10.

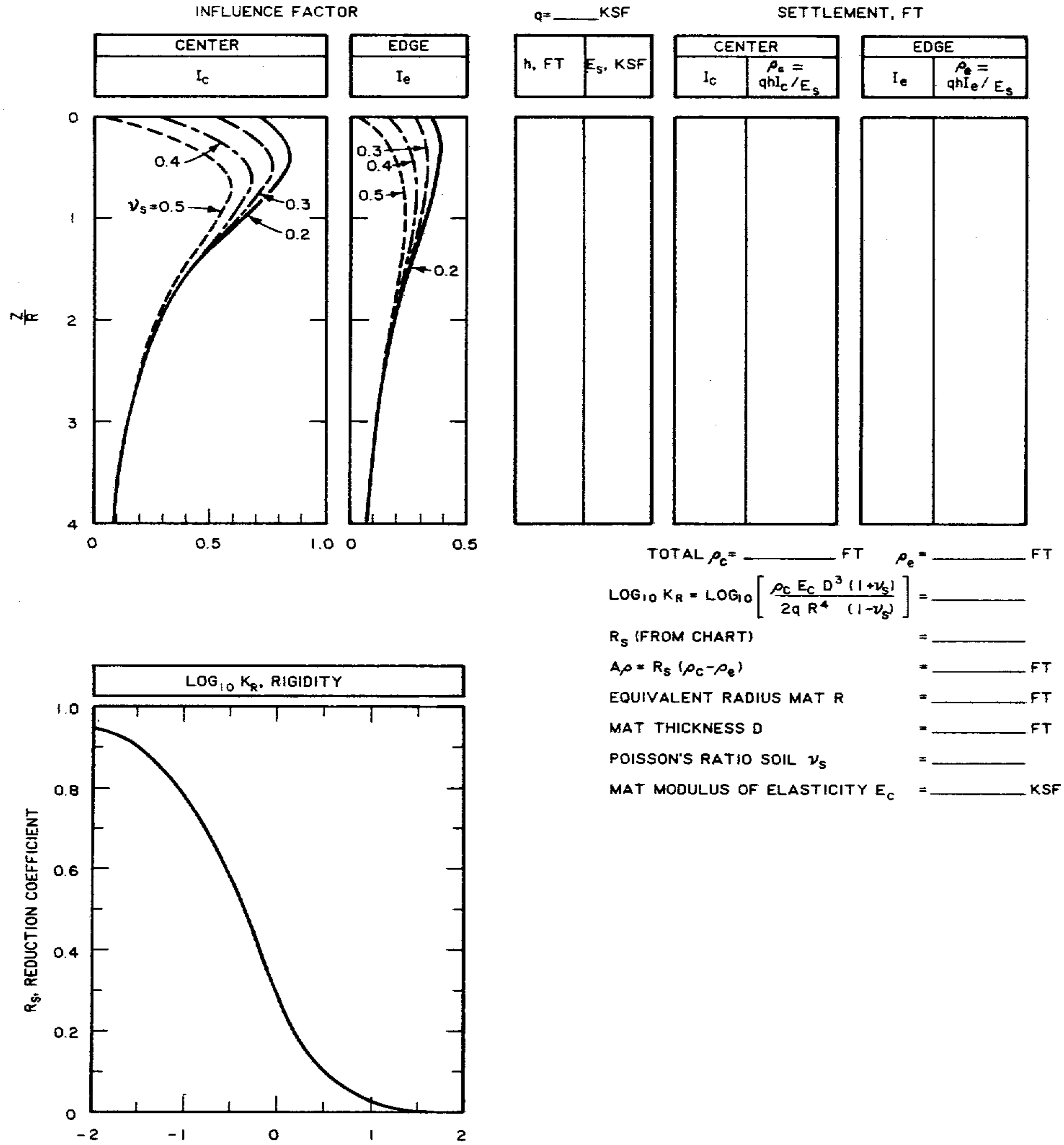


Figure 3-10. Computation of elastic settlement beneath a mat foundation (data from item 31). z = depth beneath mat, ft; R = equivalent radius , ft.

3-10. <u>Application</u>. A footing 10 ft square, 1 ft thick with base 3 ft below ground surface, is to be constructed on cohesive soil. The pressure applied on the footing is q = 2 tsf (4 ksf). The equivalent elastic modulus of this clay, which is 10 ft deep beneath the footing, is 175 tsf (350 ksf) and Poisson's ratio is 0.4. Table 3-4 compares settlement computed by the improved Janbu and Perloff methods. Refer to Figure 3-11 for application of the Kay and Cavagnaro method.

a. Average settlement by the improved Janbu method is 0.48 inch.

Table 3-4

Estimation of Immediate Settlement for
Example Application in Cohesive Soil

Method	Equation	Immediate Settlement, ρ_i
Janbu (item 9)	3-17	D/B = 0.3, H/B = L/B = 1.0 μ_o = 1.00, μ_1 = 0.35 $\rho_i = 1.00 \cdot 0.35 \cdot 2.0 \cdot 10/175$ $= 0.040$ ft or 0.48 inch
Perloff (item 52)	3-18	From Fig. 3-9, $I_e = 0.4$, $I_c = 0.7$ Edge: $\rho_i = 0.4 \bullet 2 \bullet 10 \bullet \frac{1 - 0.16}{175}$ $= 0.038$ *ft or* 0.46 *inch* Center: $\rho_i = 0.7 \bullet 2.0 \bullet 10 \bullet \frac{1 - 0.16}{175}$ $= 0.067$ *ft or* 0.81 *inch*

b. The Kay and Cavagnaro method in Figure 3-11 calculates smaller edge settlement of 0.33 inch compared with 0.46 inch and smaller center settlement of 0.73 inch compared with 0.81 inch calculated from the Perloff method. Actual differential settlement when considering stiffness of the footing is only about 0.02 inch, Figure 3-11; the footing is essentially rigid. Settlement will be less than 1 inch and expected to be about 0.5 inch.

Section III. Primary Consolidation Settlement

3-11. Description. Vertical pressure σ_{st} from foundation loads transmitted to a saturated compressible soil mass is initially carried by fluid or water in the pores because water is relatively incompressible compared with that of the soil structure. The pore water pressure u_{we} induced in the soil by the foundation loads is initially equal to the vertical pressure σ_{st} and it is defined as excess pore water pressure because this pressure exceeds that caused by the weight of water in the pores. Primary consolidation begins when water starts to drain from the pores. The excess pressure and its gradient decrease with time as water drains from the soil causing the load to be gradually carried by the soil skeleton. This load transfer is accompanied by a decrease in volume of the soil mass equal to the volume of water drained from the soil. Primary consolidation is complete when all excess pressure has dissipated so that $u_{we} = 0$ and the increase in effective vertical stress in the soil $\Delta\sigma' = \sigma_{st}$. Primary consolidation settlement is usually determined from results of one-dimensional (1-D) consolidometer tests. Refer to Appendix E for a description of 1-D consolidometer tests.

a. Normally Consolidated Soil. A normally consolidated soil is a soil which is subject to an in situ effective vertical overburden stress σ'_o equal

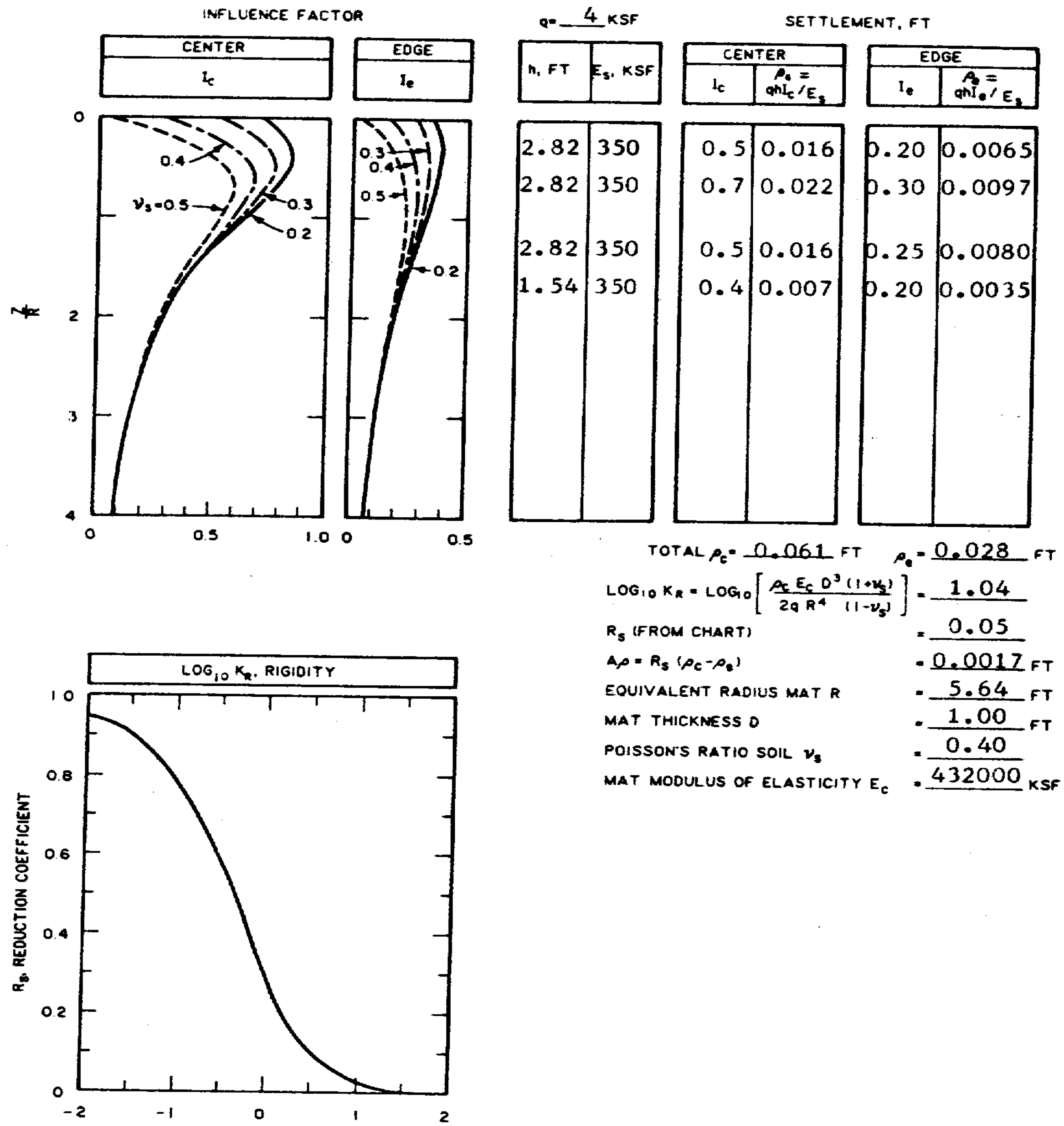


Figure 3-11. Estimation of immediate settlement for the example problem by the Kay and Cavagnaro method

to the preconsolidation stress σ'_p . Virgin consolidation settlement for applied stresses exceeding σ'_p can be significant in soft and compressible soil with a skeleton of low elastic modulus such as plastic CH and CL clays, silts, and organic MH and ML soils.

b. <u>Overconsolidated Soil</u>. An overconsolidated soil is a soil which is subject to an in situ effective overburden stress σ'_o less than σ'_p . Consolidation settlement will be limited to recompression from stresses applied to the soil up to σ'_p . Recompression settlement is usually much less than virgin consolidation settlement caused by applied stresses exceeding σ'_p .

3-12. Ultimate 1-D Consolidation. The ultimate or long-term 1-D consolidation settlement is initially determined followed by adjustment for overconsolidation effects. Refer to Table 3-5 for the general procedure to determine ultimate settlement by primary consolidation.

a. Evaluation of Void Ratio-Pressure Relationship. Estimates of the ultimate consolidation settlement following complete dissipation of hydrostatic excess pressure requires determination of the relationship between the in situ void ratio and effective vertical stress in the soil. The loading history of a test specimen taken from an undisturbed and saturated soil sample, for example, may be characterized by a void ratio versus logarithm pressure diagram, Figure 3-12.

(1) Correction of laboratory consolidation curve. Removal of an impervious soil sample from its field location will reduce the confining pressure, but tendency of the sample to expand is restricted by the decrease in pore water pressure. The void ratio will tend to remain constant at constant water content because the decrease in confinement is approximately balanced by the decrease in water pressure; therefore, the effective stress remains constant in theory after Equation 1-1 and the void ratio should not change. Classical consolidation assumes that elastic expansion is negligible and the effective stress is constant during release of the in situ confining pressure after the sample is taken from the field. Some sample disturbance occurs, however, so that the laboratory consolidation curve must be corrected as shown in Figure 3-12. Perfectly undisturbed soil should indicate a consolidation curve similar to line e_oED , Figure 3-12a, or line e_oBFE , Figure 3-12b. Soil disturbance increases the slope for stresses less than the preconsolidation stress illustrated by the laboratory consolidation curves in Figure 3-12. Pushing undisturbed samples into metal Shelby tubes and testing in the consolidometer without removing the horizontal restraint helps maintain the in situ horizontal confining pressure, reduces any potential volume change following removal from the field, and helps reduce the correction for sample disturbance.

(2) Normally consolidated soil. A normally consolidated soil in situ will be at void ratio e_o and effective overburden pressure σ'_o equal to the preconsolidation stress σ'_p . e_o may be estimated as the initial void ratio prior to the test if the water content of the sample had not changed during storage and soil expansion is negligible. In situ settlement from applied loads is determined from the field virgin consolidation curve.

(a) Reconstruction of the field virgin consolidation curve with slope C_c shown in Figure 3-12a may be estimated by the procedure in Table 3-6a. Determining the point of greatest curvature for evaluation of the preconsolidation stress requires care and judgment. Two points may be selected bounding the probable location of maximum curvature to determine a range of probable preconsolidation stress. Higher quality undisturbed specimens assist in reducing the probable range of σ'_p . If σ'_p is greater than σ'_o , then the soil is overconsolidated and the field virgin consolidation curve should be reconstructed by the procedure in Table 3-6b. The scale of the plot may have some influence on evaluation of the parameters.

(b) Consolidation settlement may be estimated by

$$\rho_{cj} = \frac{\Delta e_j}{1 + e_{oj}} \bullet H_j \tag{3-20}$$

Table 3-5

Procedure for Calculation of Ultimate Primary Consolidation Settlement of a Compressible Stratum

Step	Description
1	Evaluate the preconsolidation stress σ'_p from results of a one-dimensional (1-D) consolidation test on undisturbed soil specimens using the Casagrande construction procedure, Table 3-6a, or by methods in paragraph 1-5a. Refer to Appendix E for a description of 1-D consolidation tests.
2	Estimate the average initial effective overburden pressure σ'_o in each compressible stratum using soil unit weights, depth of overburden on the compressible stratum, and the known groundwater level or given initial pore water pressure in the stratum. Refer to Equation 1-1, $\sigma'_{oz} = \gamma z - u_w$. $\sigma'_o = (\sigma'_{oz1} + \sigma'_{oz2})/2$ where σ'_{oz1} = effective pressure at top of compressible stratum and σ'_{oz2} = effective pressure at bottom of compressible stratum.
3	Determine the soil initial void ratio e_o as part of the 1-D consolidation test or by methods in Appendix II, EM 1110-2-1906, Laboratory Soils Testing.
4	Evaluate the compression index C_c from results of a 1-D consolidation test using the slope of the field virgin consolidation line determined by the procedure in Table 3-6a as illustrated in Figures 3-12 and 3-13, or preliminary estimates may be made from Table 3-7. Determine the recompression index C_r for an overconsolidated soil as illustrated in Figures 3-12 and 3-13; preliminary estimates may be made from Figure 3-14.
5	Estimate the final applied effective pressure σ'_f where $\sigma'_f = \sigma'_o + \sigma_{st}$. σ_{st} , soil pressure caused by the structure, may be found from Equation C-2 or Boussinesq solution in Table C-1.
6	Determine the change in void ratio Δe_j of stratum j for the pressure increment $\sigma'_f - \sigma'_o$ graphically from a data plot similar to Figure 3-12, from Equation 3-21 for a normally consolidated soil, or from Equation 3-23 for an overconsolidated soil.
7	Determine the ultimate one-dimensional consolidation settlement of stratum j with thickness H_j , from Equation 3-20 $$\rho_{cj} = \frac{\Delta e_j}{1 + e_{oj}} H_j$$
8	Determine the total consolidation ρ_c of the entire profile of compressible soil from the sum of the settlement of each stratum, Equation 3-22

Table 3-5. Concluded

Step	Description
	$$\rho_c = \sum_{j=1}^{n} \rho_{cj}$$
9	Correct ρ_c for effect of overconsolidation and small departures from 1-D compression on the initial excess pore pressure using the Skempton and Bjerrum procedure, Equation 3-24 $$\rho_{\lambda c} = \lambda \rho_c$$ where λ is found from Figure 3-15. $\lambda = 1$ if $B/H > 4$ or if depth to the compressible stratum is > 2B. The equivalent dimension of the structure when corrected to the top of the compressible stratum B_{cor} is found by the approximate distribution $B_{cor} = (B'L')^{0.5}$ where $B' = B + z$ and $L' = L + z$, B = foundation width, L = foundation length and z = depth to top of the compressible soil profile. Substitute B_{cor} for B in Figure 3-15. $\rho_{\lambda c}$ is the corrected consolidation settlement. This correction should not be applied to bonded clays.

where

ρ_{cj} = consolidation settlement of stratum j, ft
Δe_j = change in void ratio of stratum j, $e_{oj} - e_{fj}$
e_{oj} = initial void ratio of stratum j at initial pressure σ'_{oj}
e_{fj} = final void ratio of stratum j at final pressure σ'_{fj}
H_j = height of stratum j, ft

The final void ratio may be found graphically using the final pressure σ'_f illustrated in Figure 3-12a. The change in void ratio may be calculated by

$$\Delta e_j = C_c \bullet \log_{10} \frac{\sigma'_{fj}}{\sigma'_{oj}} \tag{3-21}$$

where C_c is the slope of the field virgin consolidation curve or compression index. Figure 3-13 illustrates evaluation of C from results of a 1-D consolidation test. Table 3-7 illustrates some empirical correlations of C_c with natural water content, void ratio, and liquid limit. Refer to Chapter 3, TM 5-818-1, for further estimates of C_c.

(c) Total consolidation settlement ρ_c of the entire profile of compressible soil may be determined from the sum of the settlement of each stratum

$$\rho_c = \sum_{j=1}^{n} \rho_{cj} \tag{3-22}$$

where n is the total number of compressible strata. This settlement is considered to include much of the immediate elastic compression settlement ρ_i, Equation 3-1.

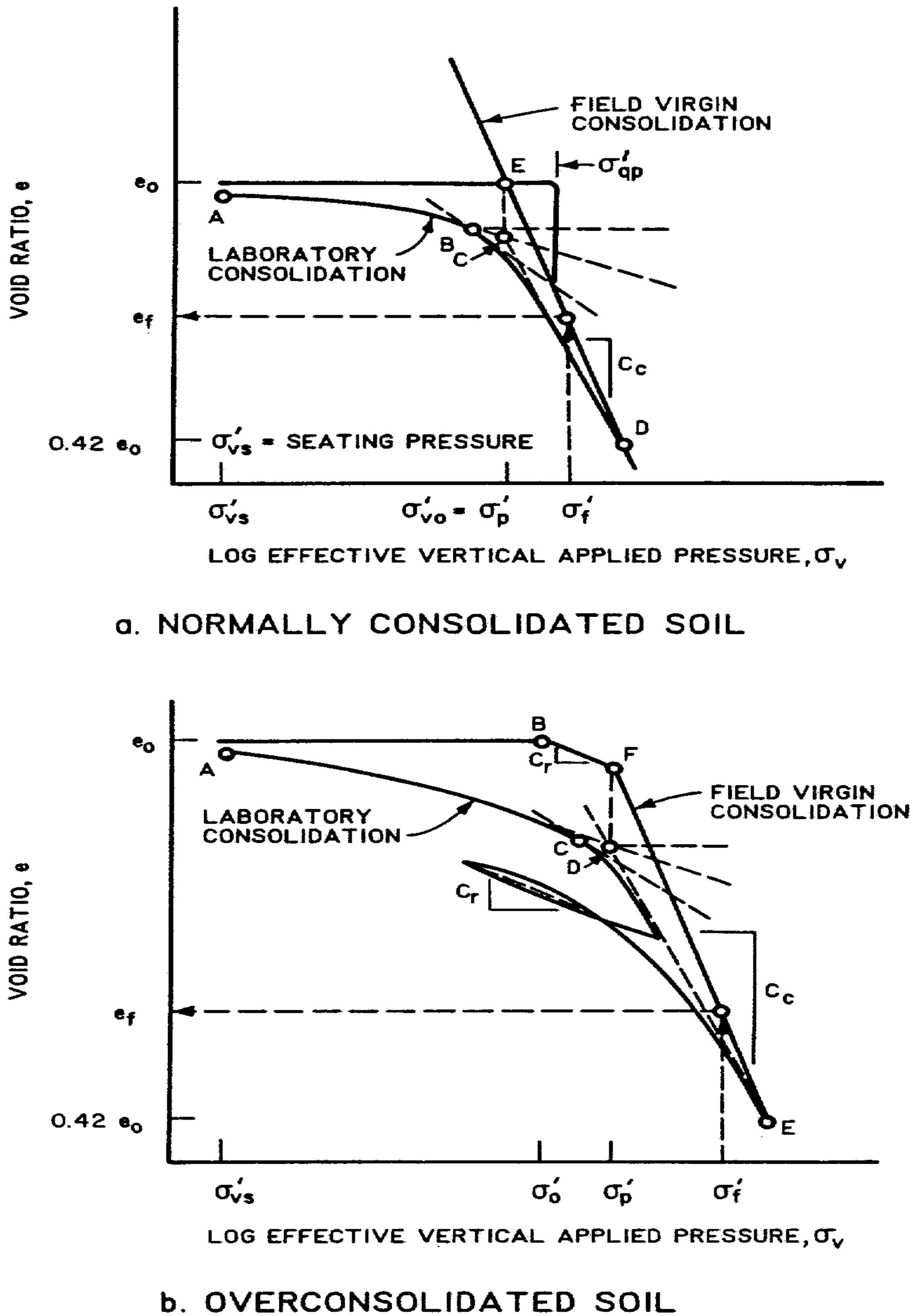


Figure 3-12. Construction of field virgin consolidation relationships

Table 3-6

Reconstruction of Virgin Field Consolidation (Data from Item 54)

a. Normally Consolidated Soil (Figure 3-12a)

Step	Description
1	Plot point B at the point of maximum radius of curvature of the laboratory consolidation curve.
2	Plot point C by the Casagrande construction procedure: (1) Draw a horizontal line from B ; (2) Draw a line tangent to the laboratory consolidation curve through B ; and (3) Draw the bisector between horizontal and tangent lines. Point C is the intersection of the straight portion of the laboratory curve with the bisector. Point C indicates the maximum past pressure σ'_p .
3	Plot point E at the intersection e_o and σ'_p . e_o is given as the initial void ratio prior to testing in the consolidometer and σ'_p is found from step 2.
4	Plot point D at the intersection of the laboratory virgin consolidaation curve with void ratio $e = 0.42e_o$.
5	The field virgin consolidation curve is the straight line determined by points E and D.

b. Overconsolidated Soil (Figure 3-12b)

Step	Description
1	Plot point B at the intersection of the given e_o and the initial estimated in situ effective overburden pressure σ'_o .
2	Draw a line through B parallel to the mean slope C_r of the rebound laboratory curve.
3	Plot point D using step 2 in Table 3-6a above for normally consolidated soil.
4	Plot point F by extending a vertical line through D up through the intersection of the line of slope C_r extending through B.
5	Plot point E at the intersection of the laboratory virgin consolidation curve with void ratio $e = 0.42e_o$.
6	The field virgin consolidation curve is the straight line through points F and E.

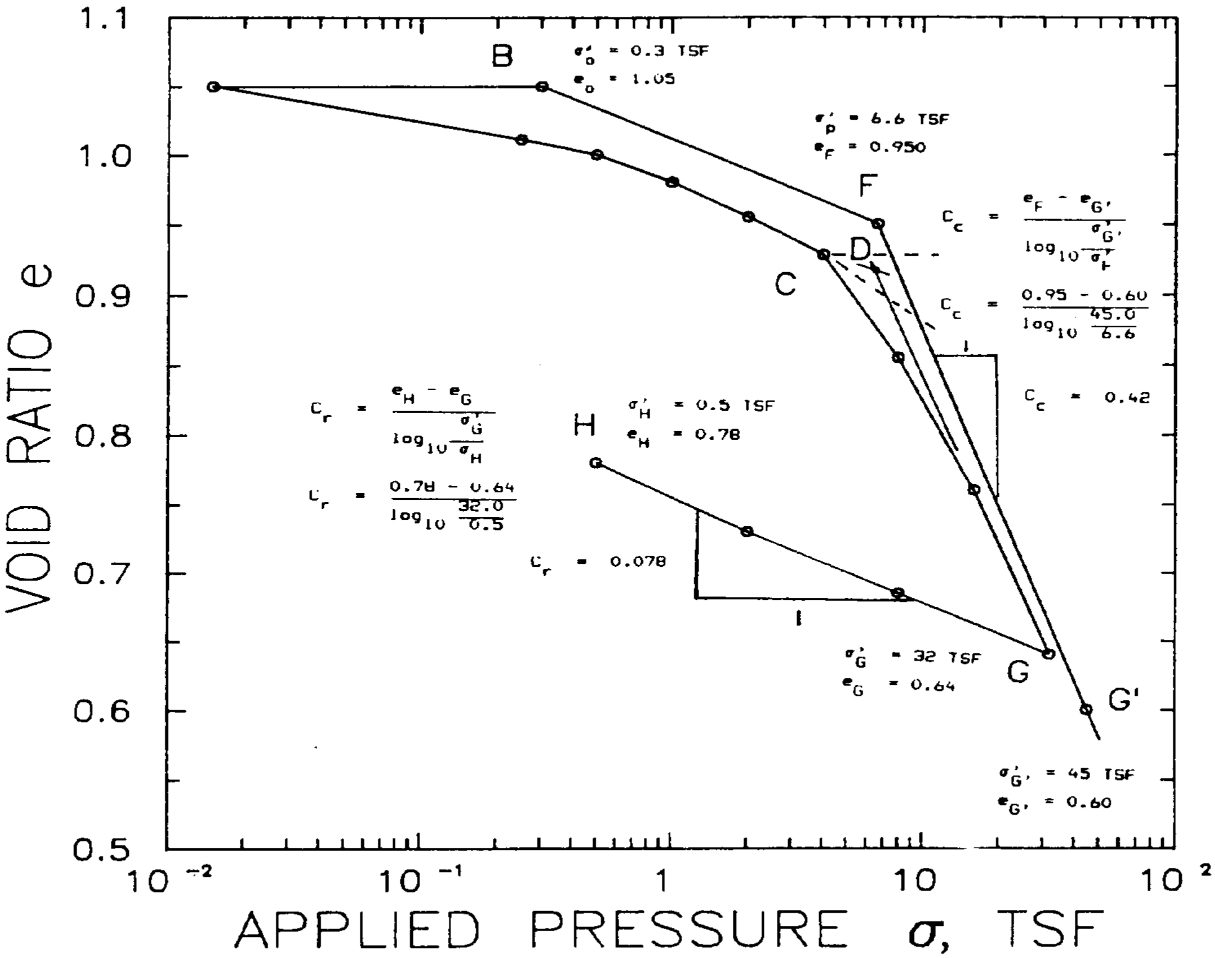


Figure 3-13. Example void ratio - logarithm pressure relationship.

(3) Apparent preconsolidation. A presumably normally consolidated soil may exhibit an apparent preconsolidation stress σ'_{qp} , Figure 3-12a. σ'_{qp} may be caused by several mechanisms; for example, the most common cause is secondary compression or the gradual reduction in void ratio (accompanied by an increase in attractive force between particles) at constant effective stress over a long time. Other causes of σ'_{qp} include a change in pore fluid, which causes attractive forces between particles to increase, or cementation due to precipitation of cementatious materials from flowing groundwater. This apparent preconsolidation is sensitive to strain and may not be detected because of sample disturbance. Existence of σ'_{qp} in the field can substantially reduce settlement for a given load and can be used to reduce the factor of safety or permit greater pressures to be placed on the foundation soil, provided that collapse will not be a problem. Refer to Chapter 5-7 to 5-10 for estimating potential collapse.

(4) Overconsolidated soil. An overconsolidated soil will be at a void ratio e_o and effective vertical confining pressure σ'_o represented by point B , Figure 3-12b. At some time in the past the soil was subject to the preconsolidation stress σ'_p , but this pressure was later reduced, perhaps by soil erosion or removal of glacial ice, to the existing overburden pressure σ'_o . The in situ settlement for an applied load will be the sum of recompression settlement between points B and F and any virgin consolidation from a

Table 3-7

Estimates of the Virgin Compression Index, C_c

Soil	C_c
Organic soils with sensitivity less than 4	0.009(LL - 10)
Organic soils, peat	$0.0115W_n$
Clays	$1.15(e_o - 0.35)$ $0.012W_n$ 0.01(LL - 13)
Varved clays	$(1 + e_o)\cdot[0.1 + 0.006(W_n - 25)]$
Uniform silts	0.20
Uniform sand	
Loose	0.05 to 0.06
Dense	0.02 to 0.03

Note: LL = liquid limit, percent
W_n = natural water content, percent
e_o = initial void ratio

final effective vertical applied pressure σ'_f exceeding the preconsolidation stress σ'_p . Reloading a specimen in the consolidometer will give the laboratory curve shown in Figure 3-12b.

(a) Reconstruction of the field virgin consolidation curve with slope C_c may be estimated by the procedure in Table 3-6b. Refer to Table 3-7 for methods of estimating C_c .

(b) The rebound loop in the laboratory curve is needed to develop the recompression line BF . Evaluation of the recompression index C_r is illustrated in Figure 3-13. The recompression index is equal to or slightly smaller than the swelling index, C_s . Approximate correlations of the swelling index are shown in Figure 3-14.

(c) Settlement ρ_{cj} of stratum j in inches may be estimated as the sum of recompression and virgin consolidation settlements. The final void ratio is found graphically from Figure 3-12b. The change in void ratio may be calculated by

$$\Delta e_j = C_r \bullet \log_{10}\frac{\sigma'_{pj}}{\sigma'_{oj}} + C_c \bullet \log_{10}\frac{\sigma'_{fj}}{\sigma'_{pj}} \tag{3-23}$$

where C_r is the average slope of the recompression line BF . If $\sigma'_{fj} < \sigma'_{pj}$,ignore the right-hand term of Equation 3-23 containing C_c and substitute σ'_{fj} for σ'_{pj} in the term containing C_r . Settlement of stratum j is

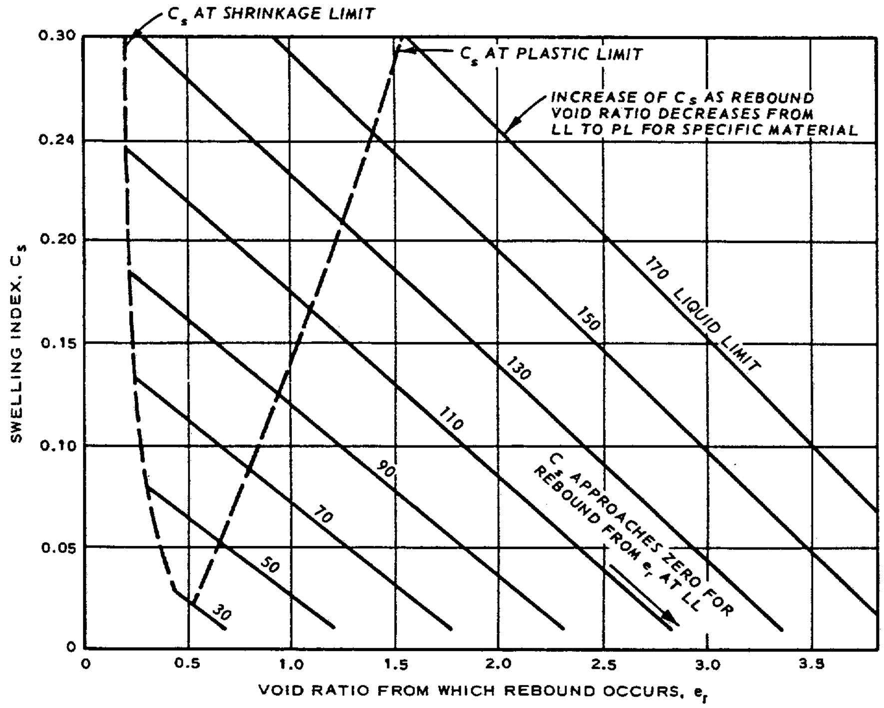


Figure 3-14. Approximate correlations for the swelling index of silts and clays (Figure 3-10, TM 5-818-1)

found from Equation 3-20 and ultimate settlement ρ_c of compressible soil in the profile is found from Equation 3-22.

(5) Underconsolidated soil. Occasionally, a compressible soil stratum may be found to have excess hydrostatic pore pressures such as when the stratum had not reached equilibrium pore water pressures under existing overburden pressures or the groundwater level had been lowered. The effective stress will increase as the pore pressures dissipate and cause recompression settlement until the effective stress equals the preconsolidation stress. Virgin consolidation settlement will continue to occur with increasing effective stress until all excess pore pressures are dissipated. If the initial effective stress is less than the preconsolidation stress σ'_p, then the ultimate settlement may be found as for an overconsolidated soil from Equations 3-23 and 3-20. σ'_{oj} is the initial effective stress found from Equation 1-1, the initial total overburden pressure minus the initial total pore water pressure. σ'_{fj} is the final effective stress found from the final total overburden pressure minus the equilibrium or final pore water pressure. If σ'_{oj} equals σ'_p, then the ultimate settlement may be found as for a normally consolidated soil from Equations 3-21 and 3-20.

b. Adjustment for Overconsolidation Effects. The effects of overconsolidation and departure from 1-D compression on the initial excess pore pressure may require correction to the calculated settlement and rate of settlement. The following semi-empirical procedures have been used to correct for these effects. Numerical methods of analysis offer a rational alternative approach to include 3-D affects, but these have not proved useful in practice.

(1) Skempton and Bjerrum correction. The corrected consolidation settlement $\rho_{\lambda c}$ of a clay stratum is found by

$$\rho_{\lambda c} = \lambda \rho_c \tag{3-24}$$

where λ is the settlement correction factor, Figure 3-15. The equivalent dimension of the loaded area should be corrected to the top of the compressible stratum by the approximate stress distribution method as illustrated in step 9, Table 3-5, or Appendix C. The corrected settlement is still assumed 1-D, although overconsolidation effects are considered. $\lambda = 1$ if $B/H > 4$ or if depth to the compressible stratum is $> 2B$.

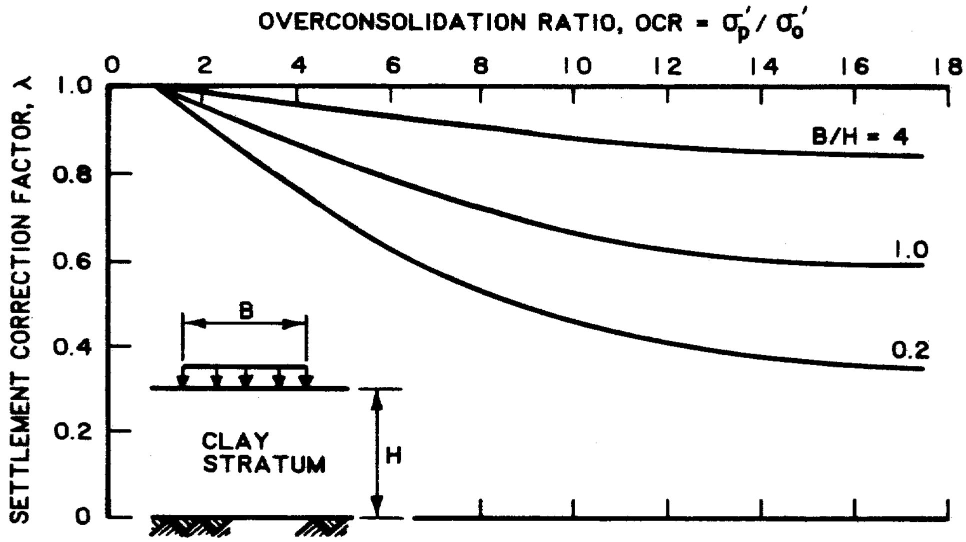


Figure 3-15. Settlement correction factor for overconsolidation effects. Reprinted by permission of the Transportation Research Board, National Research Council from Special Report 163, 1976, "Estimating Consolidation Settlement of Shallow Foundations On Overconcolidated Clay," by G. A. Leonards, p. 15.

(2) Stress path correction. This alternative approach attempts to simulate stress paths that occur in the field, as illustrated in Table 3-8. This procedure may require special laboratory tests using triaxial cells capable of undrained loading followed by consolidation. These tests have not usually been performed and are without standard operating guidelines. Approximations necessary to estimate suitable points in the soil profile for testing and estimates of stresses applied to soil elements at the selected points may introduce errors more significant than the Skempton and Bjerrum correction procedure.

3-13. Time Rate of Settlement. The solution for time rate of primary consolidation settlement is based on the Terzaghi 1-D consolidation theory in which settlement as a function of time is given by

$$\rho_{ct} = \frac{U_t \bullet \rho_{\lambda c}}{100} \tag{3-25}$$

where

Table 3-8

Summary of the Stress Path Procedure (Data from Item 35)

Step	Procedure
1	Select one or more points within the soil profile beneath the proposed structure.
2	Determine initial stresses and pore pressures at the selected points.
3	Estimate for each point the stress path for loading to be imposed by the structure. The stress path usually depends on undrained loading initially, followed by consolidation.
4	Perform laboratory tests which follow the estimated stress paths; duplicate initial stresses, measure strains from undrained loading, then consolidate to the final effective stress σ_f' and measure strains.
5	Use the strains measured to estimate settlement of the proposed structure.

ρ_{ct} = consolidation settlement at time t , ft
U_t = degree of consolidation of the compressible stratum at time t , percent
$\rho_{\lambda c}$ = ultimate consolidation settlement adjusted for overconsolidation effects, ft

Refer to Table 3-9 for the general procedure to determine time rate of settlement from primary consolidation.

a. Evaluation of the Degree of Consolidation. Solution of the Terzaghi consolidation theory to determine U_t is provided in Table 3-10 as a function of time factor T_v for four cases of different distributions of the initial excess pore water pressure. Figure 3-16 illustrates example distributions of the initial excess pore water pressure for single (drainage from one surface) and double (drainage from top and bottom surfaces) drainage.

(1) Time factor. The time factor is given by

$$T_v = \frac{C_v t}{H_e^2} \tag{3-26}$$

where

c_v = coefficient of consolidation of the stratum, ft/day
H_e = equivalent height of the compressible stratum, ft

Table 3-9

Time Rate of Settlement

Step	Description
1	Evaluate lower and upper bound values of the coefficient of consolidation, c_v , of each soil stratum in the profile for each consolidation load increment from deformation-time plots of data from 1-D consolidometer tests. Plot c_v as a function of the logarithm of applied pressure. Refer to Table 3-11 and Figure 3-17 for methods of calculating c_v .
2	Select appropriate values of c_v from the c_v versus logarithm pressure plots using σ'_f found from step 5, Table 3-5. Preliminary estimates of c_v may be made from Figure 3-18.
3	Select minimum and maximum values of c_v and calculate the effective thickness H' of a multilayer soil profile using the procedures in Table 3-12 relative to one of the soil layers with a given c_{vi} . If the soil profile includes pervious incompressible seams, then evaluate T_v and U_t in steps 4 to 6 for each compressible layer and calculate U_t of the soil profile by step 7.
4	Evaluate minimum and maximum time factors T_v of the compressible soil profile from Equation 3-26 $$T_V = \frac{c_V t}{H_e^2}$$ for various times t using c_v from step 3 (or c_{vi} for multilayer soil). The equivalent compressible soil height H_e is 1/2 of the actual height (or 1/2 of the effective height H' of multiple soil layers) for double drainage from top and bottom surfaces of the compressible soil and equal to the height of the compressible soil for single drainage.
5	Select the case, Table 3-10 and Figure 3-16 that best represents the initial pore water pressure distribution. If none of the given pressure distributions fit the initial distribution, then approximate the initial distribution as the sum or difference of some combination of the given standard distributions in Table 3-10 as illustrated in Figure 3-19. Note the cases and relative areas of the standard pore water pressure distributions used to approximate the initial distribution.
6	Evaluate minimum and maximum values of the degree of consolidation U_t for given T_v from Table 3-10. If none of the four cases in Table 3-10 model the initial pore pressure distribution, then the overall degree of consolidation may be evaluated by dividing the pore pressure distribution into areas that may be simulated by the cases in Table 3-10 and using Equation 3-28

Table 3-9. Concluded

Step	Description
	$$U_t = \frac{U_{t1}A_1 + U_{t2}A_2 + \ldots U_{ti}A_i}{A}$$ where U_{ti} = degree of consolidation of case i , i = 1 to 4; A_i = area of pore pressure distribution of case i; A = area of approximated pore pressure distribution. U_t may also be the degree of consolidation of a soil bounded by internal drainage layers (pervious soil). Omit step 7 if U_t is the degree of consolidation of the soil where pervious seams are not present.
7	Evaluate influence of internal drainage layers (pervious seams) on settlement by, Equation 3-29 $$U_t = \frac{1}{\rho_c} \bullet (U_{t1}\rho_{c1} + U_{t2}\rho_{c2} + \ldots U_{tn}\rho_{cn})$$ where U_t is the degree of consolidation at time t and ρ_c is the ultimate consolidation settlement of the compressible soil profile. Subscripts 1, 2, ..., n indicate each compressible layer between seams.
8	Determine the consolidation settlement as a function of time ρ_{ct} , where $\rho_{ct} = U_t \cdot \rho_{\lambda c}$, Equation 3-25.

The equivalent thickness of a compressible stratum for single drainage (drainage from one boundary) is the actual height of the stratum. H_e is 1/2 of the actual height of the stratum for double drainage (drainage from top and bottom boundaries).

(2) Coefficient of consolidation. The coefficient of consolidation c_v may be found experimentally from conventional (step load) laboratory 1-D consolidometer test results by four methods described in Table 3-11, Figure 3-17 and Appendix E. Both Casagrande and Taylor methods, Table 3-11, are recommended and they may provide reasonable lower and upper bound values of the coefficient of consolidation. The Casagrande logarithm time method is usually easier to use with the less pervious cohesive soils; whereas, the Taylor square root of time method is easier to use with the more pervious cohesionless soils.

Table 3-10

Degree of Consolidation as a Function of Time Factor T_v

T_v	Average Degree of Consolidation, U_t Percent)			
	Case 1*	Case 2	Case 3	Case 4
0.004	7.14	6.49	0.98	0.80
0.008	10.09	8.62	1.95	1.60
0.012	12.36	10.49	2.92	2.40
0.020	15.96	13.67	4.81	4.00
0.028	18.88	16.38	6.67	5.60
0.036	21.40	18.76	8.50	7.20
0.048	24.72	21.96	11.17	9.69
0.060	27.64	24.81	13.76	11.99
0.072	30.28	27.43	16.28	14.36
0.083	32.51	29.67	18.52	16.51
0.100	35.68	32.88	21.87	19.77
0.125	39.89	36.54	26.54	24.42
0.150	43.70	41.12	30.93	28.86
0.175	47.18	44.73	35.07	33.06
0.200	50.41	48.09	38.95	37.04
0.250	56.22	54.17	46.03	44.32
0.300	61.32	59.50	52.30	50.78
0.350	65.82	64.21	57.83	56.49
0.400	69.79	68.36	62.73	61.54
0.500	76.40	76.28	70.88	69.95
0.600	81.56	80.69	77.25	76.52
0.800	88.74	88.21	86.11	85.66
1.000	93.13	92.80	91.52	91.25
1.500	98.00	97.90	97.53	97.45
2.000	99.42	99.39	99.28	99.26

*See Figure 3-16

(a) The Casagrande logarithm time method, Figure 3-17a, determines

$$c_v = \frac{0.197 h_e^2}{t_{50}} \tag{3-27a}$$

where

c_v = coefficient of consolidation of stratum, ft^2/day
h_e = equivalent specimen thickness, ft
t_{50} = time at 50 percent of primary consolidation, days

The equivalent specimen thickness is the actual specimen height for single drainage and 1/2 of the specimen height for double drainage. This method usually provides a low value or slow rate of consolidation.

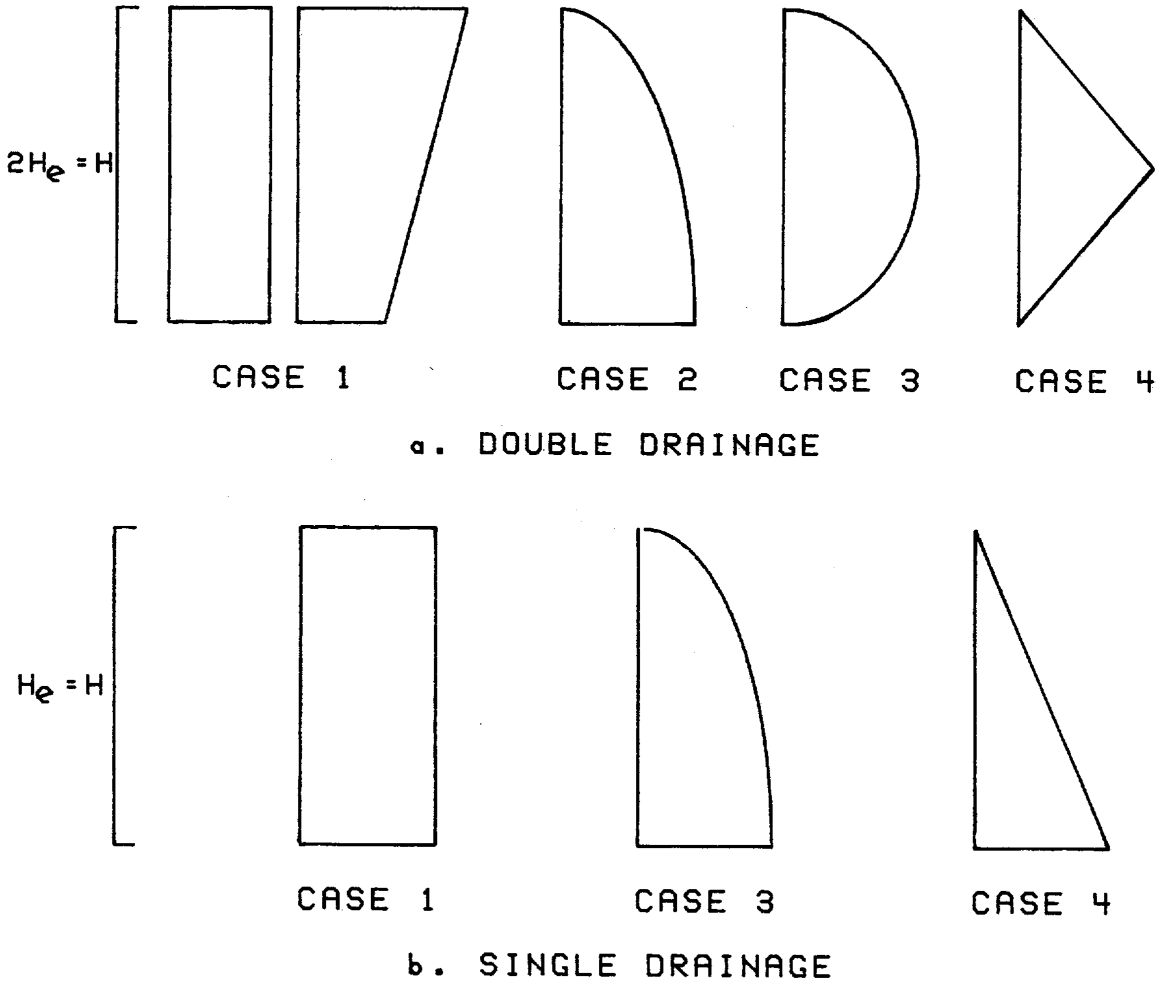


Figure 3-16. Example distributions of excess pore water pressure for double and single drainage. H is the actual stratum thickness and H_e is the equivalent height.

(b) The Taylor square root of time method, Figure 3-17b, determines

$$c_v = \frac{0.848 h_e^2}{t_{90}} \tag{3-27b}$$

This method usually calculates a faster rate of consolidation than the Casagrande method and may better simulate field conditions.

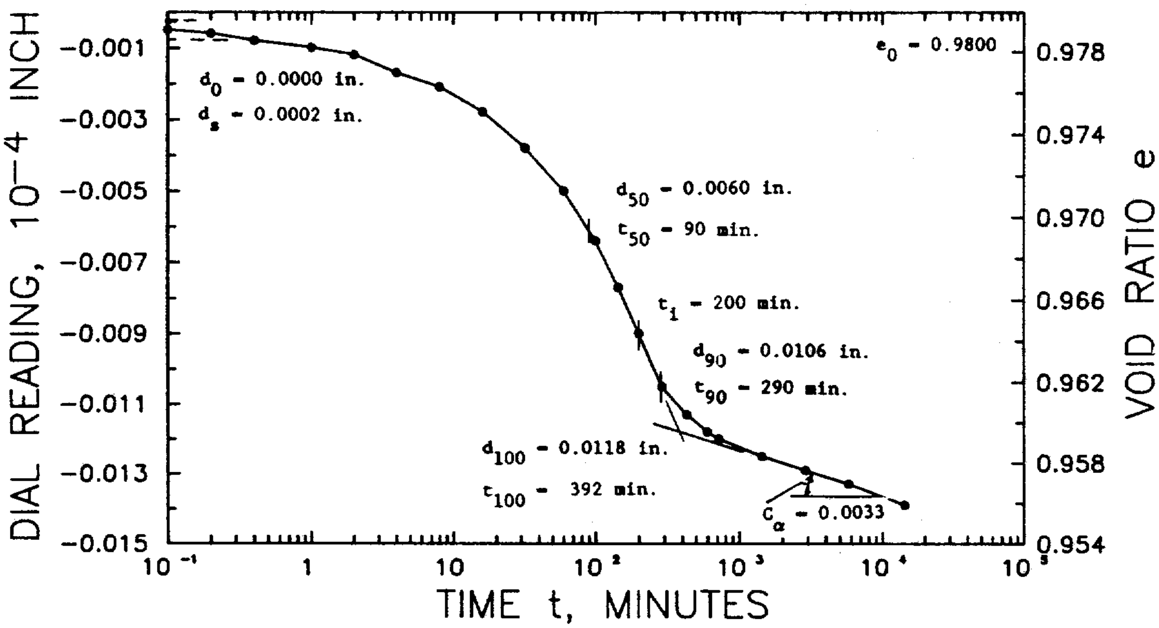
(c) c_v should be plotted as a function of the applied consolidation pressure. An appropriate value of c_v can be selected based on the final effective pressure σ_f' of the soil for a specific case.

(d) Figure 3-18 illustrates empirical correlations of the coefficient of consolidation with the liquid limit.

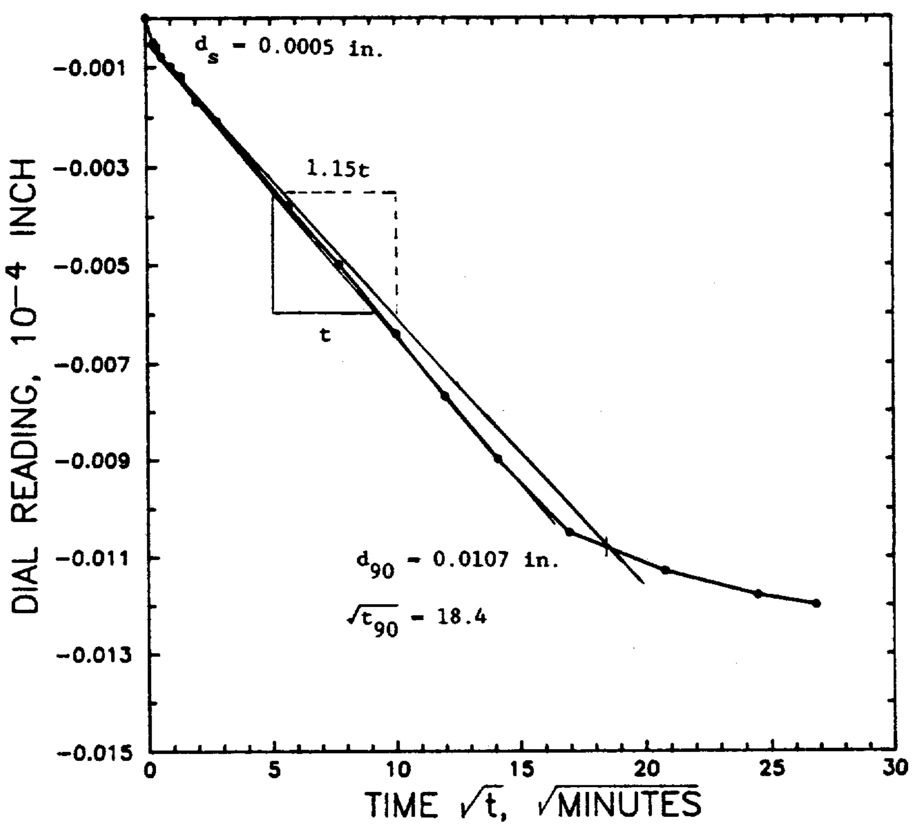
Table 3-11

Evaluation of the Coefficient of Consolidation c_v by the Step Load Test (Data from Item 66)

Method	Equation for c_v	Procedure	Example
Terzaghi	$\frac{0.848h_e^2}{t_{90}}$	1. Measure initial specimen height h_o and set initial dial reading d_o.	h_o = 1.148 inches d_o = 0.0000 inch h_f = 1.140 inches d_s = 0.0002 inch Refer to Figure 3-17a
		2. Measure dial reading d as a function of time t and final specimen height h_f. Plot d versus $\log_{10}t$. Determine d_s, the corrected zero point ($d_o - d_s$ = initial compression), by measuring vertical distance between time of about 0.1 min. and a time that is 4 times this.	
		3. Determine time t_{100} and compression d_{100} to 100 percent consolidation as the intersection of the tangent and asymptote of the consolidation curve. Then determine d_{90} and t_{90}.	t_{100} = 392 min. or 0.27 day d_{100} = 0.0118 in. $d_{90} = d_s + 0.9(d_{100} - d_s)$ = 0.0002 + 0.9(0.0118-0.0002) = 0.0002 + 0.0104 = 0.0106 inch. t_{90} = 290 min. or 0.20 day from Figure 3-17a
		4. Determine equivalent thickness of drainage path, $h_e = (h_o + h_f)/2$ if single drainage; $h_e = (h_o + h_f)/4$ if double drainage	$h_e = (h_o + h_f)/4$ = 2.288/4 = 0.572 inch or 0.0477 ft
		5. Calculate c_v	$c_v = \frac{0.85 \cdot 0.0477^2}{0.20} = 0.010\ ft^2/day$
Casagrande	$\frac{0.197h_e^2}{t_{50}}$	Same as Terzaghi above except determine time to reach 50 percent of consolidation t_{50}	$d_{50} = d_s + 0.5(d_{100} - d_s)$ = 0.0002 + 0.5(0.0118-0.0002) = 0.0002 + 0.0058 = 0.0060 inch t_{50} = 90 min. or 0.063 day $c_v = \frac{0.197 \cdot 0.0477}{0.063} = 0.007\ ft^2/day$
Inflection Point	$\frac{0.405h_e^2}{t_i}$	Same as above except note time to reach inflection point of curve t_i	t_i = 200 min. or 0.14 day from Figure 3-17a $c_v = \frac{0.405 \cdot 0.0477^2}{0.14} = 0.007\ ft^2/day$
Taylor	$\frac{0.848h_e^2}{t_{90}}$	1. Measure initial specimen height h_o and set initial dial reading d_o	h_o = 1.148 inches d_o = 0.0000 inch h_f = 1.140 Refer to Figure 3-17b
		2. Measure dial reading d as a function of time t and final specimen height h_f. Plot d versus $\sqrt{t}$ or square root of time.	
		3. Extend straight line portion back to $\sqrt{t} = 0$ to obtain corrected initial reading d_o.	d_o = 0.0005 inch from Figure 3-17b
		4. Through d_o draw a straight line with inverse slope 1.15 times tangent and intersect laboratory curve to obtain t_{90}	$\sqrt{t_{90}}$ = 18.4 or t_{90} = 339 min. or 0.23 day Refer to Figure 3-17b
		5. Determine h_e as above	h_e = 0.0477 ft
		6. Calculate c_v	$c_v = \frac{0.848 \cdot 0.0477^2}{0.23} = 0.008\ ft^2/day$



a. VOID RATIO-LOGARITHM TIME



b. DISPLACEMENT-SQUARE ROOT TIME

Figure 3-17. Example time plots from 1-D consolidometer test, $\Delta\sigma$ = 1 TSF

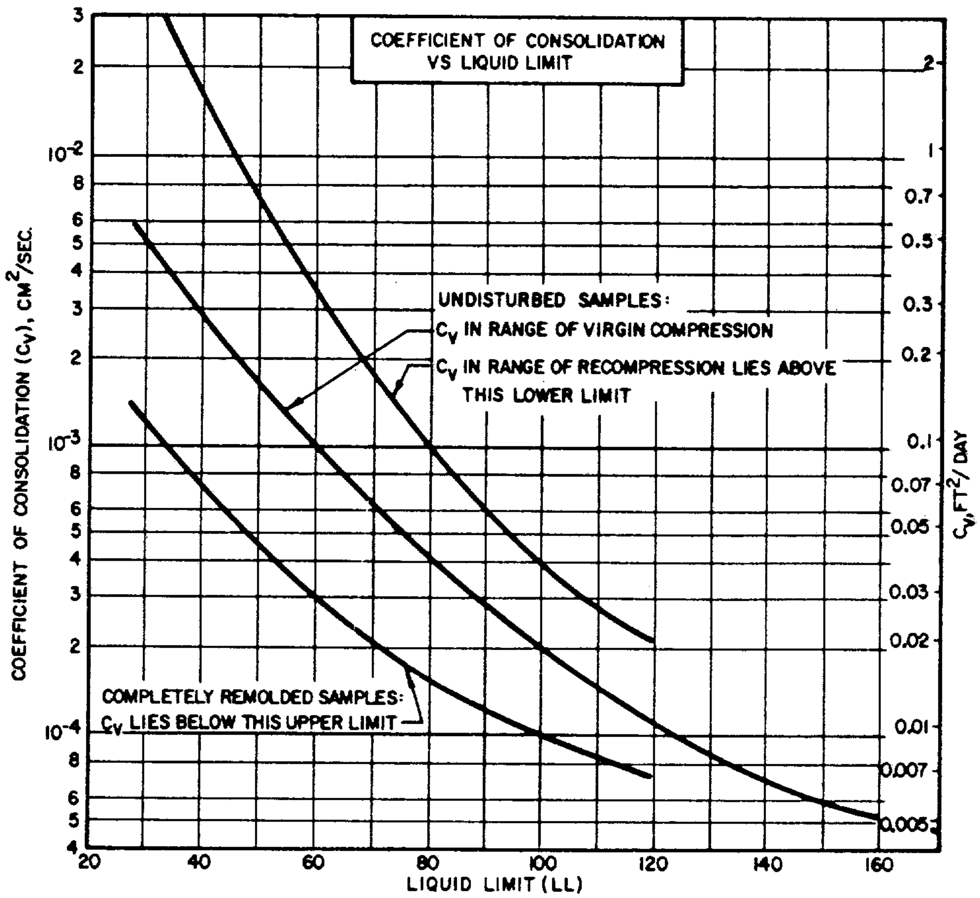


Figure 3-18. Correlations between coefficient of consolidation and liquid limit (NAVFAC DM 7.1)

(e) The procedure shown in Table 3-12 should be used to transform a compressible soil profile with variable coefficients of consolidation to a stratum of equivalent thickness H' and coefficient of consolidation c_v . T_v may be calculated from Equation 3-26 with $H_e = H'$. Refer to 3-13d, "Internal Drainage Layers", to estimate U_t of a soil profile with pervious incompressible sand seams interspersed between compressible soil.

b. Superposition of Excess Pore Pressure Distribution. An initial pore pressure distribution that is not modeled by any of the four cases in Table 3-10 and Figure 3-16 may sometimes be approximated by superposition of any of the four cases and the overall or weighted degree of consolidation found by

$$U_t = \frac{U_{t1}A_1 + U_{t2}A_2 + \ldots U_{ti}A_i}{A} \tag{3-28}$$

where A represents the areas of the initial pore pressure distributions. Subscripts 1, 2, ..., i indicate each pore pressure distribution. Linearity of the differential equations describing consolidation permits this assumption.

(1) Example excess pore water pressure distributions. Some example complex excess pore water pressure distributions are shown in Figure 3-19.

Table 3-12

Procedure to Evaluate the Effective Thickness and Average Degree of Consolidation for Multiple Soil Layers (After NAVFAC DM 7.1)

Step	Description
1	Select any layer i , with coefficient of consolidation c_{vi} and thickness H_i
2	Transform the thickness of every other layer to an effective thickness H'_j $H'_1 = H_1 \left[\frac{c_{vi}}{c_{v1}}\right]^{1/2}$ $H'_2 = H_2 \left[\frac{c_{vi}}{c_{v2}}\right]^{1/2}$ $H'_n = H_n \left[\frac{c_{vi}}{c_{vn}}\right]^{1/2}$
3	Calculate the total effective thickness by $H' = H'_1 + H'_2 + \ldots + H'_i + \ldots + H'_n$
4	Treat the entire thickness as a single layer of effective thickness H' with a coefficient of consolidation $c_v = c_{vi}$ and evaluate the time factor T_v from Equation 3-26. Evaluate the degree of consolidation with the assistance of Table 3-10 and Figure 3-16.

(2) Application. For single drainage a decreasing excess pore pressure distribution may be modeled as illustrated in Figure 3-19b. If $T_v = 0.2$, the degree of consolidation is 50.41 and 37.04 percent for cases 1 and 4, respectively, Table 3-10. The overall degree of consolidation from Equation 3-27 for the example in Figure 3-19b is

$$U_t = \frac{50.41 \bullet H_e \bullet H_o - 37.04 \bullet 0.5 \bullet H_e \bullet H_o/2}{H_e \bullet H_o - 0.5 \bullet H_e \bullet H_o/2}$$

$$U_t = \frac{50.41 \bullet 1.0 - 37.04 \bullet 0.25}{1.0 - 0.25} = 54.87 \textit{ percent}$$

The total area of the complex pore pressure distribution equals the area of case 1 less area of case 4, Figure 3-19b.

c. Internal Drainage Layers. Internal drainage layers of pervious soil within an otherwise low permeable clay stratum will influence the rate of settlement. This influence can be considered by summation of the degrees of consolidation of each compressible layer between the pervious seams by (item 52)

$$U_t = \frac{1}{\rho_c} \bullet (U_{t1} \bullet \rho_{c1} \bullet + U_{t2} \bullet \rho_{c2} + \ldots + U_{tn} \bullet \rho_{cn}) \qquad (3\text{-}29)$$

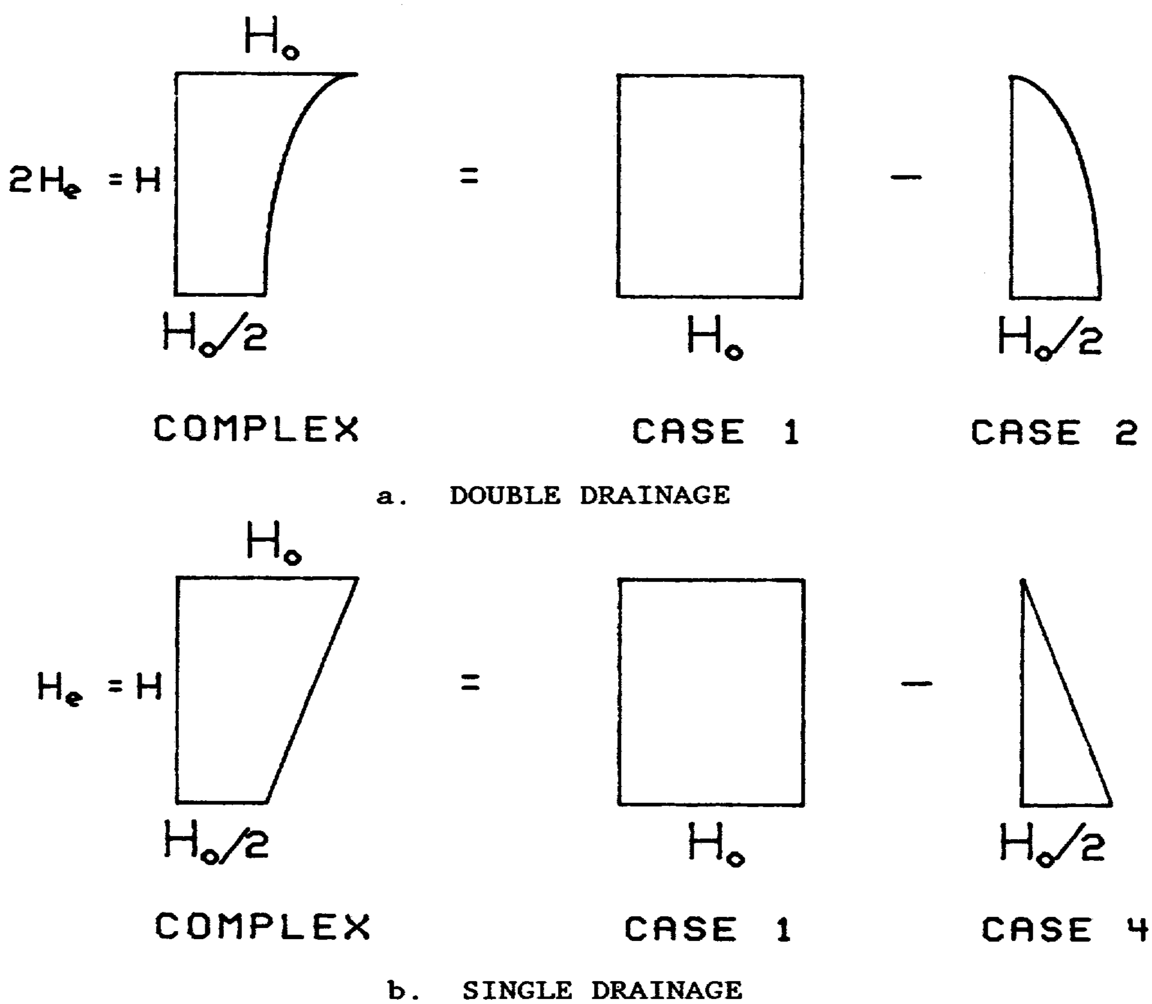
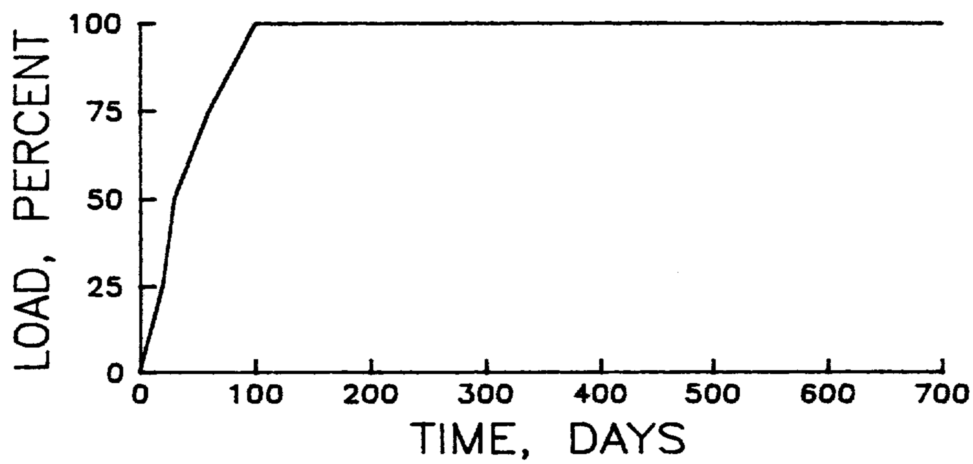


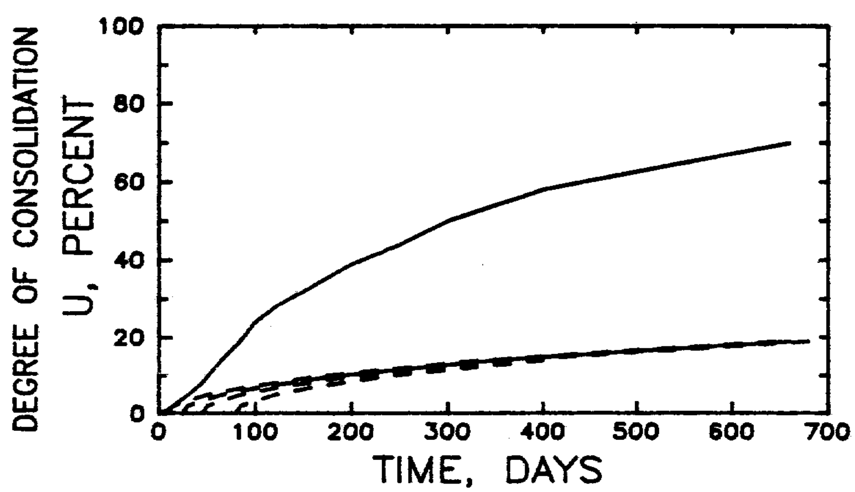
Figure 3-19. Example complex excess pore water pressure distributions

where U_t is the degree of consolidation at time t and ρ_c is the ultimate consolidation settlement of the entire compressible stratum. The subscripts 1, 2, ..., n indicate each compressible layer between pervious seams.

d. Time-Dependent Loading. The rate of load application to foundation soils is usually time-dependent. Estimates of the degree of consolidation of time-dependent loads may be made by dividing the total load into several equal and convenient increments such as the 25 percent increments illustrated in Figure 3-20. Each increment is assumed to be placed instantaneously at a time equal to the average of the starting and completion times of the placement of the load increment. The degree of consolidation U of the underlying compressible soil is evaluated for each of the equal load increments as a function of time and divided by the number of load increments to obtain a weighted U. Only one curve need be evaluated for the soil if the thickness of the compressible stratum and coefficient of consolidation are constant. The weighted U of each load increment may then be summed graphically as illustrated in Figure 3-20 to determine the degree of consolidation of the time-dependent loading. Chapter 5 of NAVFAC DM 7.1 provides a nomograph for evaluating U for a uniform rate of load application.



a. TIME-DEPENDENT LOADING

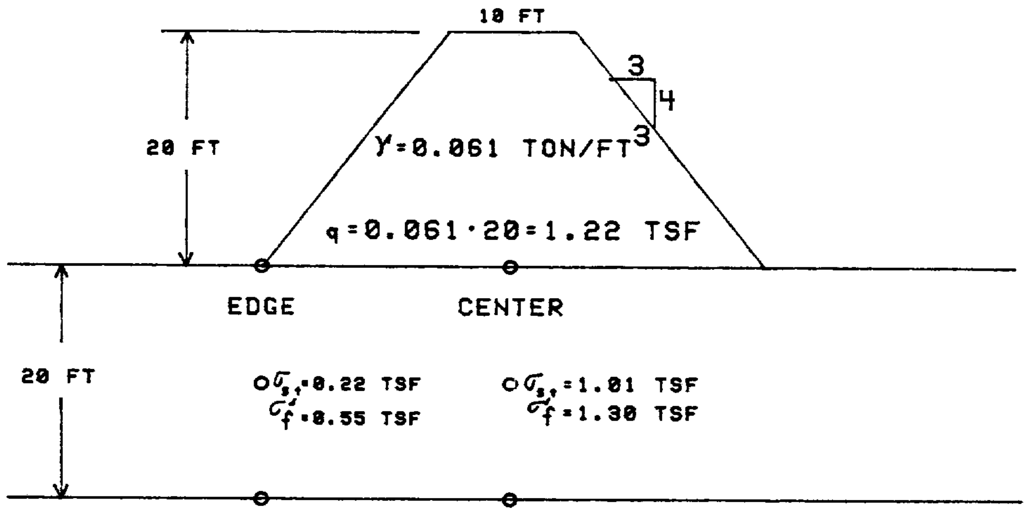


b. DEGREE OF CONSOLIDATION

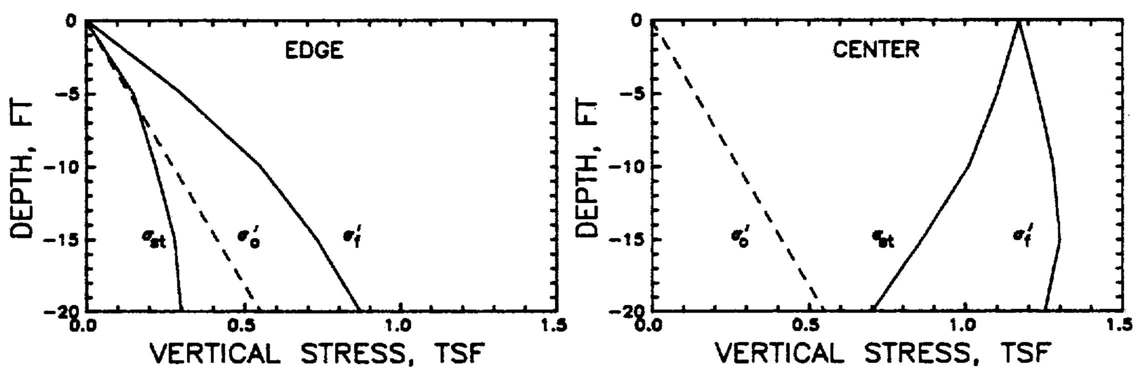
Figure 3-20. Degree of consolidation for time-dependent loading

3-14. <u>Example Application of Primary Consolidation</u>. An embankment, Figure 3-21, is to be constructed on a compressible clay stratum 20 ft thick. The groundwater level is at the top of the compressible clay stratum. A consolidometer test was performed on an undisturbed specimen of the soil stratum after the standard load procedure described in EM 1110-2-1906. The specimen was taken from a depth of 10 ft and drainage was allowed on both top and bottom surfaces. A plot of the laboratory consolidation void ratio versus logarithm pressure relationship is shown in Figure 3-13.

a. <u>Ultimate Primary Consolidation</u>. The procedure described in Table 3-5 was applied to evaluate ultimate settlement beneath the edge and center of the embankment by hand calculations. The solution is worked out in Table 3-13a.



a. ELEVATION DETAIL



b. VERTICAL STRESS DISTRIBUTION

Figure 3-21. Embankment for example application

b. Time Rate of Consolidation. The procedure described in Table 3-9 was applied to evaluate the rate of settlement beneath the edge and center of the embankment by hand calculations assuming an instantaneous rate of loading. The solution is worked out in Table 3-13b.

3-15. Accuracy of Settlement Predictions. Experience shows that predictions of settlement are reasonable and within 50 percent of actual settlements for many soil types. Time rates of settlement based on laboratory tests and empirical correlations may not be representative of the field because time rates are influenced by in situ fissures, existence of high permeable sand or low permeable bentonite seams, impervious boundaries, and nonuniform soil parameters as well as the rate of construction.

a. Preconsolidation Stress. Soil disturbance of laboratory samples used for 1-D consolidation tests decreases the apparent preconsolidation stress.

Table 3-13

Evaluation of Consolidation Settlement by Hand Calculations for Example Application of Embankment, Figure 3-21

a. Total Settlment

Step	Description
1	The preconsolidation stress σ'_p after the Casagrande construction procedure, Table 3-6a, is 6.6 tsf shown in Figure 3-13 (neglecting a minimum and maximum range). Since $\sigma'_p > \sigma'_o$, the soil is overconsolidated with an OCR of about 22. The field virgin consolidation line is evaluated by the procedure in Table 3-6b.
2	The initial effective stress distribution σ'_o , is shown in Figure 3-21b. The wet unit weight γ is 0.061 ton/ft^3 and γ_w is 0.031 ton/ft. σ'_o at 10 ft of depth is 0.3 tsf.
3	The initial void ratio of the specimen prior to consolidation is e_o = 1.05, Figure 3-13.
4	The virgin compression index $C_c = 0.42$ and the recompression index $C_r = 0.078$, Figure 3-13.
5	The pressure distribution applied by the embankment was calculated using the trapezoidal distribution, Table C-1b (Appendix C). At 10 ft below ground surface the vertical stress applied by the embankment at the edge is 0.22 tsf and at the center is 1.01 tsf. The final effective pressure σ'_f 10 ft below ground surface at the edge is 0.55 tsf and at the center 1.30 tsf. The estimated pressure distributions are shown in Figure 3-21b.
6	The change in void ratio Δe at the 10 ft depth may be found from Equation 3-23 where the right-hand part of the equation containing C_c is ignored because $\sigma'_f < \sigma'_p$, $\Delta e = C_r \bullet \log \frac{\sigma'_f}{\sigma'_o}$ *Edge*: $\Delta e_e = 0.078 \bullet \log_{10} \frac{0.55}{0.30} = 0.020$ *Center*: $\Delta e_c = 0.078 \bullet \log_{10} \frac{1.30}{0.30} = 0.050$
7, 8	Settlement of the stratum from Equation 3-20 is *Edge*: $\rho_{ce} = \frac{0.020}{1.00 + 1.05} \bullet 20 = 0.195 \ ft \ or \ 2.34 \ inches$ *Center*: $\rho_{cc} = \frac{0.050}{1.00 + 1.05} \bullet 20 = 0.488 \ ft \ or \ 5.85 \ inches$ Improved reliability may be obtained by testing additional specimens at different depths within the compressible stratum, calculating settlements within smaller depth increments, and adding the calculated settlements, Equation 3-22.

Table 3-13. Continued

Step	Description
9	Settlement may be corrected for overconsolidation effects after the Skempton and Bjerrum procedure, Equation 3-24. λ is about 0.8 from Figure 3-15 for an overconsolidation ratio > 18. *Edge*: $\rho_{3ce} = 0.8 \bullet 2.34 = 1.87$ *inches* *Center*: $\rho_{3cc} = 0.8 \bullet 5.85 = 4.68$ *inches*

b. Time Rate of Settlement

Step	Description
1,2	Minimum and maximum estimates of the coefficient of consolidation may be made using the methods in Table 3-11 from a plot of the deformation as a function of time data, Figure 3-17. These data indicate c_v values from 0.007 ft^2/day to 0.010 ft^2/day, Table 3-11. The range of applied consolidation pressures is from 1 to 2 tsf using double drainage during the consolidation test.
3	The time factors for the range of c_v from 0.007 to 0.010 ft^2/day is, Equation 3-26, $\frac{0.007t}{10^2} < T_v < \frac{0.010t}{10^2} = 0.00007t < T_v < 0.00010t$ where the time t is in days. The compressible stratum is assumed to drain on both top and bottom surfaces; therefore, the equivalent height H_e is 10 ft.
4	The excess pore water pressure distribution given by σ_{st} in Figure 3-21 appears to be similar to case 2 at the edge and case 1 at the center, Figure 3-16a. The average degree of consolidation in percent after 1, 10, and 50 years using Table 3-10 is

Time		T_v		U_t, Percent			
				Edge (Case 2)		Center (Case 1)	
Years	Days	Min	Max	Min	Max	Min	Max
1	364	0.025	0.036	14.7	18.8	17.1	21.4
10	3640	0.255	0.364	54.7	65.4	56.7	67.0
50	18200	1.274	3.640	95.6	98.9	95.8	98.9

Step	Description
5	Cases 1 and 2 of Figure 3-16a are considered representative of the initial excess pore water pressure distributions so that superposition of the cases in Table 3-9, step 5 and 6, is not necessary.

Table 3-13. Concluded

Step	Description
6	Consolidation settlement as a function of time t is, Equation 3-24

Time, Years	Settlement ρ_{ct}, inches			
	Edge		Center	
	Min	Max	Min	Max
1	0.27	0.35	0.80	1.00
10	1.02	1.22	2.65	3.14
50	1.79	1.85	4.48	4.63
∞	1.87		4.68	

b. Virgin Compression Index. Soil disturbance decreases the compression index.

c. Swelling and Recompression Indices. Soil disturbance increases the swelling and recompression indices.

d. Coefficient of Consolidation. Soil disturbance decreases the coefficient of consolidation for both virgin compression and recompression, Figure 3-18, in the vicinity of initial overburden and preconsolidation stresses. The value of c_v decreases abruptly at the preconsolidation stress for good undisturbed samples.

e. Field Test Embankment. A field test embankment may be constructed for significant projects to estimate field values of soil parameters such as C_c and c_v. Installation of elevation markers, inclinometers, and piezometers allow the measurement of settlement, lateral movement, and pore pressures as a function of time. These field soil parameters may subsequently be applied to full-scale structures.

3-16 Computer Solutions. Several computer programs are available to expedite calculation of settlement and rates of settlement of structures constructed on multilayer soil profiles.

a. Vertical Stress Distribution. The vertical stress distribution from Boussinesq and Westergaard solutions may be computed beneath foundations, single and multiple footings, and embankments from Program CSETT (item 61) and Program I0016 (item 45).

b. Ultimate Consolidation Settlement. Long-term consolidation settlement of structures may be computed assuming the Terzaghi 1-D consolidation by Program MAGSETTI (item 45) using output from Program CSETT or I0016.

c. Ultimate Consolidation and Rate of Settlement. One-dimensional consolidation settlement and rates of settlement by Terzaghi 1-D consolidation theory may be computed by Program FD31 (item 45) and Program CSETT (item 61).

(1) Program FD31 does not consider the influence of the vertical stress distribution with depth, and, therefore, it is applicable to fills or embankments with lateral dimensions substantially greater than the thickness of the consolidating soil.

(2) Program CSETT considers loaded regions of simple and complex geometric shapes for single or multiple and time-dependent loads. Loads may be 2- or 3-D. Stress distributions may be calculated by either Boussinesq or Westergaard methods. The program allows analysis of multiple soil layers and a variety of drainage conditions. Output consists of total settlement, settlement of individual layers, and degree of consolidation as a function of time and location requested by the user.

d. Settlement of Soft Soil. Settlement from desiccation and consolidation in soft, compressible soil with large void ratios may be computed by Program PCDDF (item 8). This program is applicable to dredged material and considers time-dependent loads, influence of void ratio on self-weight, permeability, nonlinear effective stress relationships, and large strains.

e. Settlement of Shallow Foundations in Sand. Corps Program I0030, "CSANDSET", can calculate the immediate settlement in sands of 14 different procedures including Alpan, Schultze and Sherif, Terzaghi and Peck, Schmertmann, and elastic methods. Program I0030 considers water table depth, embedment depth, and foundation dimensions for a variety of soil conditions in multilayer sands. Soil input data include SPT, CPT, elastic modulus, and water table depth.

f. Vertical Displacement of Various Soil Types. Appendix F provides a user's manual and listing of computer program VDISPL for calculating immediate settlement of granular materials using Schmertmann's procedure modified to consider prestress. Program VDISPL also can calculate immediate settlement of an elastic soil, consolidation/swell of an expansive soil, and settlement of a collapsible soil (see Chapter 5). Finite element program CON2D (item 15) may be used to calculate plane strain 2-D consolidation settlement of embankments and structures on multiple soil layers using the Cam Clay elasto-plastic constitutive soil model. CON2D may also analyze consolidation of saturated and partly saturated earth masses for time-dependent vertical loads to determine settlement, rate of settlement, and pore pressure distribution. This program may analyze the condition of saturated and partly saturated earth mass.

Section IV. Secondary Compression and Creep

3-17. Description. Secondary compression and creep are time-dependent deformations that appear to occur at essentially constant effective stress with negligible change in pore water pressure. Secondary compression and creep may be a dispersion process in the soil structure causing particle movement and may be associated with electrochemical reactions and flocculation. Although creep is caused by the same mechanism as secondary compression, they differ in the geometry of confinement. Creep is associated with deformation without volume and pore water pressure changes in soil subject to shear; whereas, secondary compression is associated with volume reduction without significant pore water pressure changes.

a. Model. Secondary compression and creep may be modeled by empirical or semi-empirical viscoelastic processes in which hardening (strengthening) or softening (weakening) of the soil occurs. Hardening is dominant at low stress levels; whereas, weakening is dominant at high stress levels. Deformation in soil subject to a constant applied stress can be understood to consist of three stages. The first stage is characterized by a change in rate of deformation that decreases to zero. The second or steady state stage occurs at a constant rate of deformation. A third stage may also occur at sufficiently large loads in which the rate of deformation increases ending in failure as a result of weakening in the soil. Soil subject to secondary compression in which the volume decreases as during a 1-D consolidometer test may gain strength or harden with time leading to deformation that eventually ceases, and, therefore, the second (steady state) and third (failure states) may never occur.

b. Relative Influence. Secondary compression and creep are minor relative to settlement caused by elastic deformation and primary consolidation in many practical applications. Secondary compression may contribute significantly to settlement where soft soil exists, particularly soft clay, silt, and soil containing organic matter such as peat or Muskeg or where a deep compressible stratum is subject to small pressure increments relative to the magnitiude of the effective consolidation pressure.

3-18. Calculation of Secondary Compression. Settlement from secondary compression ρ_s has been observed from many laboratory and field measurements to be approximately a straight line on a semi-logarithmic plot with time, Figure 3-17a, following completion of primary consolidation. The decrease in void ratio from secondary compression is

$$\Delta e_{st} = C_{\alpha} \bullet \log \frac{t}{t_{100}} \tag{3-30}$$

where

Δe_{st} = change in void ratio from secondary compression at time t
C_α = coefficient of secondary compression
t = time at which secondary compression settlement is to be calculated, days
t_{100} = time corresponding to 100 percent of primary consolidation, days

Secondary compression settlement is calculated from Equation 3-20 similar to primary consolidation settlement.

a. Coefficient of Secondary Compression. C_α is the slope of the void ratio-logarithm time plot for time exceeding that required for 100 percent of primary consolidation, t_{100} . t_{100} is arbitrarily determined as the intersection of the tangent to the curve at the point of inflection with the tangent to the straight line portion representing a secondary time effect, Figure 3-17a.

b. Estimation of C_α. A unique value of C_α/C_c has been observed, Table 3-14, for a variety of different types of soils. The ratio C_α/C_c is constant and the range varies between 0.025 and 0.100 for all soils. High values of C_α/C_c relate to organic soils. C_α will in general increase with

Table 3-14

Coefficient of Secondary Compression C_α
(Data from Item 43)

Soil	C_α/C_c
Clay	0.025 - 0.085
Silt	0.030 - 0.075
Peat	0.030 - 0.085
Muskeg	0.090 - 0.100
Inorganic	0.025 - 0.060

time if the effective consolidation pressure σ' is less than a critical pressure or the preconsolidation stress σ'_p. For σ' greater than σ'_p, C_α will decrease with time; however, C_α will remain constant with time within the range of effective pressure $\sigma' > \sigma'_p$ if C_c also remains constant (e.g., the slope of the e-log σ curve is constant for $\sigma' > \sigma'_p$). A first approximation of the secondary compression index C_α is $0.0001W_n$ for $10 < W_n < 3000$ where W_n is the natural water content in percent (after NAVFAC DM 7.1).

c. Accuracy. Soil disturbance decreases the coefficient of secondary compression in the range of virgin compression. Evaluation of settlement caused by secondary compression has often not been reliable.

d. Example Problem. The coefficient of secondary compression was determined to be 0.0033 and time t_{100} is 392 minutes or 0.27 day, Figure 3-17a, for this example problem. The change in void ratio after time t = 10 years or 3640 days is, Equation 3-30,

$$\Delta e = C_\alpha \bullet \log_{10}\frac{t}{t_{100}} = 0.0033 \bullet \log_{10}\frac{3640}{0.27} = 0.0136$$

The settlement from Equation 3-20 for an initial void ratio $e_{100} = 0.96$ is

$$\rho_s = \frac{0.0136}{1 + 0.96} \bullet 20 = 0.139 \text{ ft or } 1.67 \text{ inches}$$

for a stratum of 20-ft thickness.